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UNIVERSITÉ DE MONTRÉAL

METHODOLOGY FOR
RISK ASSESSMENT IN NATURAL HAZARDS
(FOCUS ON LANDSLIDES)

ADEL EL-RAMLAWY

DÉPARTEMENT DE GÉNIE CIVIL

ÉCOLE POLYTECHNIQUE DE MONTRÉAL

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Présenté par: EL-RAMLAWY Adel

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M. KAHAWITA René, Ph.D., président

M. SILVESTRI Vincenzo, Ph.D., membre et directeur de recherche

M. LAFLEUR Jean, Ph.D., membre

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ABSTRACT

Significant economic damage and loss of life occurs each year as a result of natural hazards. Landslide is one of those hazards that has effects on certain regions and can be caused by many factors. Historical cases have shown that thousands of lives have been lost due to landslides.

Because of the unlimited reasons and style variation in slope failure, slope failure cannot be accurately predicted. Therefore, risk assessment has been recognized as a measurement of expected hazard. The basic approaches used for risk assessment have been presented in chapter (4).

This thesis develops a simplified model for risk degree of landslide hazard. This is done by taking into account all the factors which may affect occurrence of landslide. These factors include the factor of safety of slope stability, nature related factors, human related factors, previous recurrence data, as well as mitigation measurements.

By using a mathematical model, risk factors are ranked from (1)to (12). These factors are based on the natural environment and human activities that may cause landslides. The higher the risk factor is, the higher probability of landslide occurrence. That means that a zone of risk factor equal to 10 has a higher probability of landslide than a zone of 9 as risk factor and so on. The above model has been applied to a case study in Repentigny city.

Through the risk factor and the population as well as the assets value in a certain area, we can feel the expected risk of landslide. This may help land users to take a decision for either continuing to stay, relocation to other areas, or adoption of protective measurements.

RÉSUMÉ

Des dommages économiques importants ainsi que des pertes de vie se produisent annuellement suite à des catastrophes naturelles. Le glissement de terrains est l'une de ces catastrophes qui affectent certaines régions et qui est causé par plusieurs facteurs. Des cas historiques de glissement de terrains ont engendré des milliers de pertes de vie à travers le monde.

À cause des raisons illimitées et de la variation des styles de dénivellations des pentes, ces dernières ne peuvent être prédites avec précision. L'évaluation du risque semble être une mesure qui permet de calculer les risques de catastrophes. L'approche utilisée pour évaluer le risque est présentée au chapitre 4.

Cette thèse présente un modèle simplifié d'un degré de risque des catastrophes dues aux glissements des terrains. Ceci est réalisé en tenant compte de tous les facteurs pouvant donner

lieu à un glissement de terrain, ces facteurs incluent: le facteur de sécurité de la stabilité des pentes, les facteurs reliés à la nature, les facteurs reliés à l'activité humaine, les données des répétitions antérieures de même que les mesures de mitigation.

Dans un modèle mathématique, les facteurs de risque sont classés de 1 à 12, ces facteurs sont basés sur l'environnement naturel et sur les activités humaines pouvant engendrer des glissements de terrains. Le facteur de risque le plus élevé correspond à la plus forte probabilité d'avoir un glissement de terrain. Ainsi la probabilité d'avoir un glissement de terrain dans une zone ayant un facteur de risque égal à 10 est supérieur à celle d'une zone ayant un facteur de risque égal à 9, et ainsi de suite. Le modèle ci-dessus a été appliqué sur des études de cas faites dans la ville de Repentigny.

En utilisant le facteur de risque, le nombre de population et les objets de valeurs dans certains espaces, on peut évaluer le risque attendu d'avoir un glissement de terrain. Ceci peut aider les occupants d'un terrain à prendre une décision concernant soit la demeure sur cette terre, ou la déménagement en d'autres lieux, ou l'adoption des mesures de protection.

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INTRODUCTION

Natural Hazards are those conditions or processes which may cause economic damages or loss of life in human population. They are distinguished from human environmental disturbance by the fact that they originate due to the natural environment rather than human action. Examples of natural hazards include floods, earthquakes, droughts, tornadoes, and landslides. Examples of human environmental disturbance include air pollution, water pollution or atomic radiation.

Landslide is one of those natural hazards. It is an event in which surface masses of a slope forming earth move outward and downward from their underlying and stable floors in response to the force of gravity (Flawn 1970). Direct causes (triggers) of landslide could be natural, earthquake for instance, or man-induced as in excavation at a slope toe. Brief description of slope stability analysis and factors that affect this stability are given in chapter 2.

Previous experiences show that landslides have been reasons for loss of life, destruction of buildings, roads traffic disturbance. Other negative impacts will depend on the land use of the affected zone. The costs of slope failure include both direct and indirect losses. Direct costs are associated with actual damage to structures. Indirect costs include loss of productivity, loss of tax revenue, and disturbance of infrastructures. A study, by Schuster 1978, has estimated the costs of slope failure to be on excess of 1,000 million dollars (US) in the United States only. Annual losses in Italy have been estimated at $1,140 million (Arnould and Frey, 1977). In many developing countries landslide may have a continuing and serious impact on social and economic structure and leads to disruption and misery to human lives.

Unfortunately, the science of soil mechanics cannot really cope with the unlimited reasons and style variation in slope failure. Still, slope failure cannot be accurately predicted. The continuous occurrence of landslides proves the difficulty of assessing slope failure factors. Hence, we need to deal with this hazard as one of those that cannot be completely controlled. Risk assessment in this case will provide information that may lead to better adjustment to hazard and reduction of its negative impacts.

THE PURPOSE OF THIS STUDY

The main purpose of this study is to develop an approximate and a relatively simple method for risk assessment of landslide as a natural hazard. This is done through elaboration of the natural and human factors which affect occurrence of landslide hazard. By using a mathematical model, risk factors are ranked from (1)to (12). The higher the risk factor, the higher probability of landslide occurrence. That means that a zone of risk factor equal to 10 has a higher probability of landslide than a zone of 9 as risk factor and so on.

Through the risk factor and the population as well as the assets value in a certain area, we can feel the expected risk of landslide. This may help land users to take a decision for either continuing to stay, relocation to other areas, or adoption of protective measurements. Those who are yet to plan use of a land will be able to compare between different zones from landslide point of view. After all, public authorities may use this approach for future land use planning. It will be also possible to study the value of corrective measurements versus an expected risk value. This will assist in evaluation of different alternatives including the "do nothing" choice. Other parties, like insurance agencies, may also benefit from the risk value for managing their business in sloped areas.

Chapter: 1

LANDSLIDE AS NATURAL HAZARD

Every parameter of the biosphere can be considered a hazard when it reaches an extreme event. Probability of extreme geophysical events gives indication on how often it may be occurring. When an event is so frequent that it is a part of normal conditions, it is no longer hazard. For example, the arctic weather conditions do not constitute any hazard for the Eskimo while it could be, for others, so difficult place to live in. Similarly, living in deserts does not cause any disturbance for a native of the African Sahara where it could be impossible life for others. Therefore, a natural hazard can be identified as extreme event in nature, potentially harmful to humans and occurring infrequently enough to be considered not part of the normal condition or state of the environment, but often enough to be of concern on a human time scale (Burton 1989).

Where there is no man or human work, there is no hazard. Nature and humans are two important elements to constitute a natural hazard. For example in no-man land, like North America before any settlers had arrived, earthquakes or floods would not cause any hazard. Therefore, the natural hazard can be described as an interaction of people and nature. Fig. 1 shows a proposed general procedure for study of a natural hazard.

1.1 Classification of natural hazards

Natural hazards can be classified according to their principal casual process to geophysical and biological hazards. Biological hazards are divided into floral and faunal. Whereas geophysical hazards can be divided to atmospheric (climatic and meteorological) phenomena and to geological processes. Table 1 shows a classification chart prepared by Burton and Kates, 1964.

The field of geology and geophysics can be subdivided to seismology, vulcanology and geomorphology (Burton, I. 1989). Seismology treats earthquakes and vulcanology study volcanoes. Finally, geomorphology treats erosion and landslides.

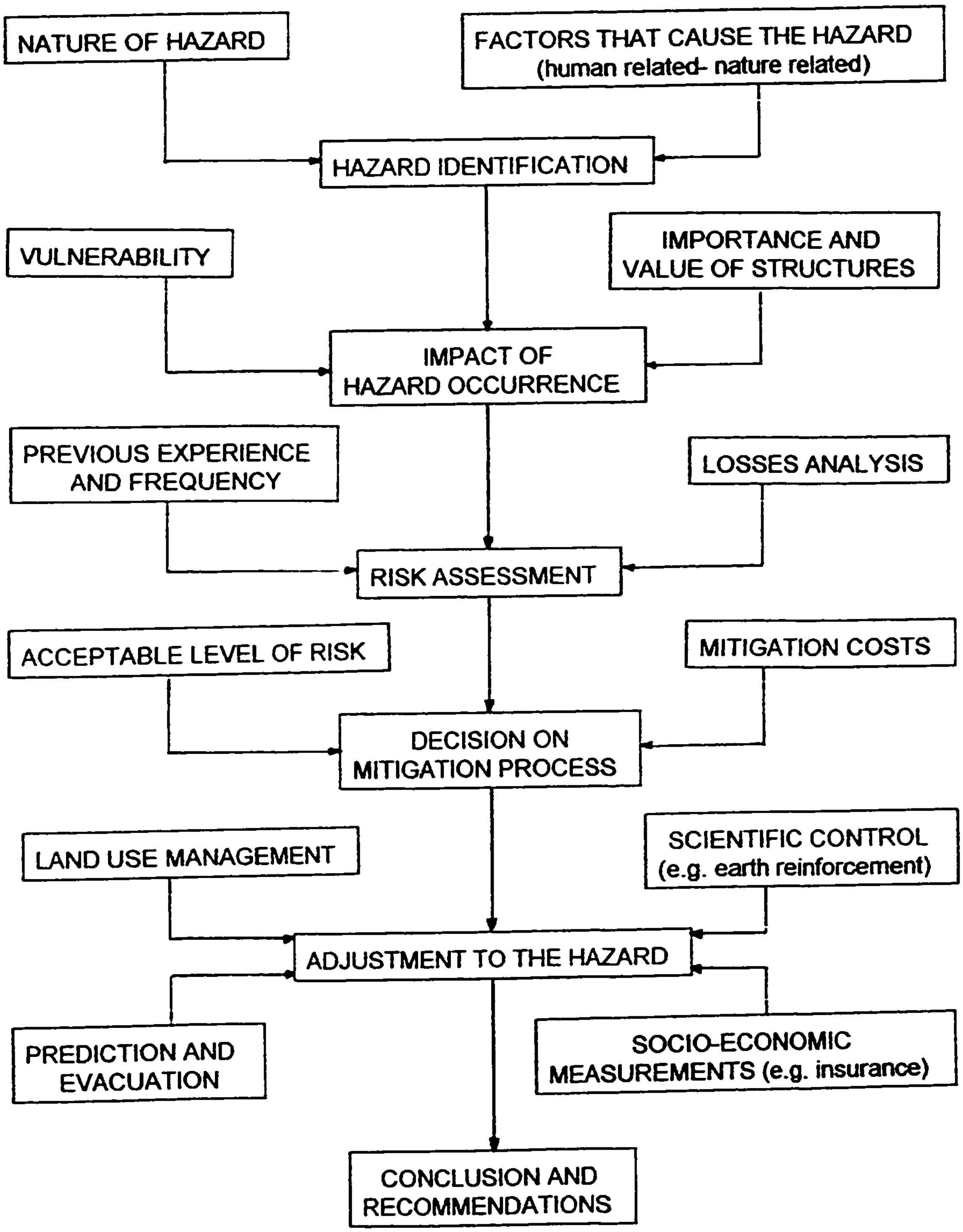


FIG. 1. STUDY OF A NATURAL HAZARD

Geophysical		Biological	
Climatic and meteorological	Geologic and geomorphic	Floral	Faunal
Snow and ice Droughts Floods Fog Frost Hail Heat waves Tropical cyclones Lightning and fire Tornadoes	Avalanches Earthquakes Erosion (including soil erosion and shore and beach erosion) Landslides Shifting sand Tsunamis Volcanic	Fungal disease (for example, athlete's foot, Dutch elm disease, wheat stem disease, rust) infestation (for example, weeds, phreatophytes, water hyacinth) Hay fever poisonous plants	Bacterial and viral diseases (for example, influenza, malaria, smallpox, rabies) Infestations (for example, of rabbits, termites, locusts) Venomous animal bites

Table 1. Classification of Natural Hazards by principal and casual agent

(After Burton and Kates, 1964)

1.2 Measurement of natural hazards

There are two main approaches to measure a natural hazard. The first is directed to the physical process. Extreme events can be measured in terms of energy release per unit of area or per unit of time as in the case of Richter scale of earthquakes measurements. Richter's scale is a complex logarithmic scale expressing the amount of displacement of the seismograph pen and the distance from the earthquake's epicenter. The amount of displacement of the pen reflects the amount of energy released as transmitted by the seismic waves.

The second approach is to measure the hazard in respect of its impacts upon man and his works. An example is the modified Mercalli scale for earthquake measurements. This scale measures the intensity of earthquakes as experienced. The measure of intensity according to the modified Mercalli scale is the ability to sense the earthquake without instrument, to observe its effects by ordinary people, and to describe its impact on people and their possessions. The measurement of earthquakes as measured by the Modified Mercalli number reflects both the characters of human settlement in the earthquake zone and the strength of the earthquake.

Good scales of measurement for the impact of natural hazards are not an easy task. The best available basis is economic loss measurement.

1.3 Landslide accidents

Individual slope failures are generally not so spectacular or so costly as earthquakes, major floods, hurricanes or some other natural disasters. Yet, they are more widespread, and over the years may cause more economic losses than any other geologic hazard. Moreover, much of the damages occurring with earthquakes and intense storms are due to landslides. For example, in the May 1970 earthquake of Peru which took about 70,000 lives, about 20,000 were killed after landslide of the north peak of Nevado Huascaran (Varnes, D.J. 1984).

One of the worst landslide disasters occurred in China in 1786. The slide was triggered by the earthquake of Kangding

Louding in Sichuan Province where about 100,000 people were killed (Turner, 1996). Another catastrophic landslide was the one in Colombia, South America in 1985. The huge slide has resulted in killing of more than 20,000 and 5,000 more were injured (Schuster, 1996).

The landslide of may 1971 in Saint-Jean-Vianny may be considered as one of the most important landslides ever to have occurred in Quebec. Roads, a bus, and 40 houses disappeared into the crater. Thirty one people died and the town was subsequently evacuated (Tavenas, F. 1971). Another example from Quebec is the Nicolet landslide of November 1955. A large section of the land slipped into the Nicolet River. The movement carried away a school and number of houses and three people died.

1.4 Risk Assessment Approach

Previous approaches for study of the problem of landslides used to depend on the calculation of the factor of safety. That is the ratio between the stabilizing strength and the driving force, tending to cause the slope failure, calculated from the geotechnical point of view. The factor of safety reflects the stability of slope.

In this approach, the factor of safety , like the other geotechnical parameters, is a fixed value. However, the fact that all such parameters exhibit a degree of uncertainty will result in a similar uncertainty in the factor of safety (Tabba, M. M. 1984). This uncertainty results from the imperfect idealization of the actual behavior by the analytical method. In addition, the factor of safety normally represents the case in a limited area. Whereas the risk assessment can introduce a global view of the problem.

The use of the conventional factor of safety as the only guide for slope failure possibility was not sufficient to avoid landslides disasters. Landslide may occur in areas with a fairly acceptable factor of safety. That is due to other factors like earthquake, flood or man made excavation at the slope toe. Recent development in landslide studies has introduced analytical and probabilistic methods, landslide risk mapping, instrumentation and slide warning systems. Countries, like France and Switzerland, have already started landslide zonation.

In USA, Krohn and Slosson (1976) have developed a landslide intensity map to define landslide prone area within the continental United states. This map is based on three principal factors: topography, bedrock, and precipitation. (White et al. 1975) has concluded, in their report "Assessment of Research on Natural Hazards", that future research needs to focus on what can be done usefully from a technological and regulatory sense, as well as how to help local governments and public at risk to implement new approaches.

In Quebec, (Rissmann et al, 1985) have completed a project for zones exposed to landslides along Yamaska river between Yamaska and Saint-Hyscinthe. Classification was basically according to topographic geometry and instability signs.

In this thesis, we try to develop a simplified model for risk degree of landslide hazard. This is done by taking into account all the factors which may affect occurrence of landslide. These factors include the factor of safety of slope stability, nature related factors, human related factors, previous recurrence data, as well as mitigation measurements.

By knowing the degree of expected risk and population prone to landslide, we can get a risk evaluation. This evaluation will help local governments and publics to implement safer land use management and to take adequate measurements to ensure acceptable level of risk.

Chapter: 2

SLOPE STABILITY PARAMETERS

Schuster (1978) has shown that the term "landslide" is the popular name, although somewhat inaccurate, for mass movement. Where mass movement is the downward and outward movement of slope forming materials. In fact, landslide is just a form of mass movement. However, the term landslide, as used, comprises almost all varieties of mass movements on slopes including some, such as rock falls, topples and debris flows, that involve little or true sliding (Varnes, D. J. 1984).

2.1 Factor of Safety (F.S)

Slope movements occur when shear stress exceed shear strength of the materials forming the slope. The ratio of shear strength to shear stress along a given surface in a slope is known as a factor of safety.

$$\text{Factor of Safety } (F) = \frac{\text{Shear Strength}}{\text{Shear Stress required for equilibrium}}$$

In which: Shear Strength = $c + \sigma \tan \phi$

This relation is represented by Mohr Circle as shown in figure 2-1. Where;

c = Cohesion intercept of Mohr Coulomb Strength diagram,

ϕ = Angle of internal friction of soil,

σ = Normal stress on strip surface.

The surface with the lowest (F) ratio is the critical surface and its factor of safety governs the stability of the slope. It is clear that the higher the factor of safety, the more stabilized slope we have. On the other hand, if the factor of safety is less than unity, slope is not supposed to exist.

2.2 Slope stability analysis

In order to devise the factors that affect factor of safety, we need to study slope stability. There are various methods available for slope analysis. The majority of these may be classified as limit equilibrium methods. Duncan (1996) has prepared a summary of characteristics of the commonly used methods. This summary is shown in table 2-1.

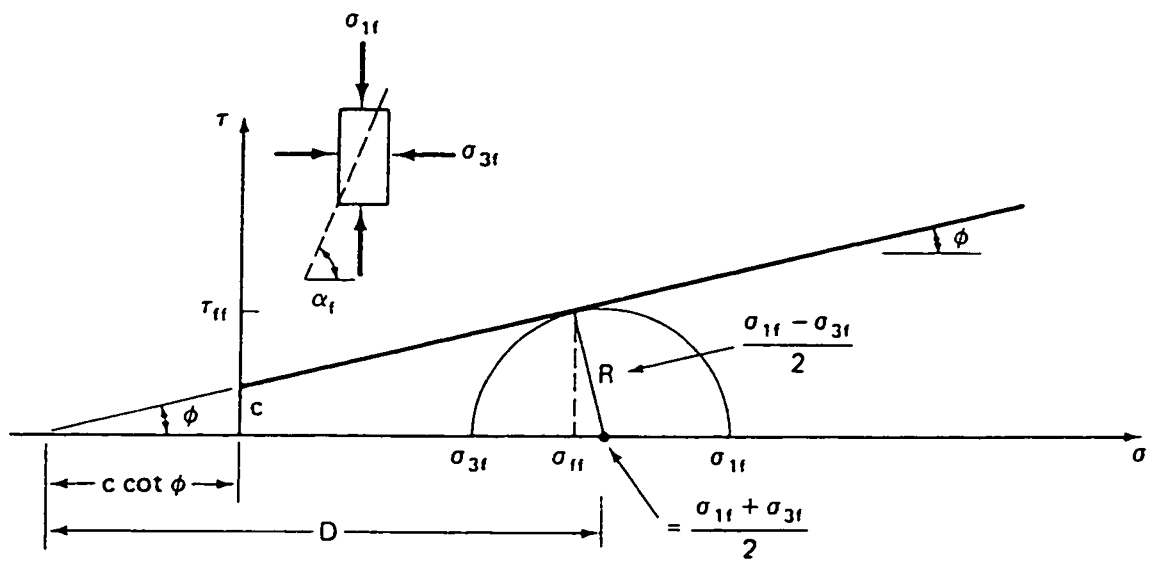


Fig. 2.1. Mohr-Coulomb strength envelope with one Mohr circle at failure.

Table 2-1. Characteristics of Commonly Used Methods for Slope Stability (After Duncan 1996)

METHOD	LIMITATIONS, ASSUMPTIONS, AND EQUILIBRIUM CONDITIONS SATISFIED
Ordinary method of slice (Fellenius1927)	Factors of safety low, very inaccurate for flat slopes with high pore pressures; only for circular slip surfaces; assumes that normal force on the base of each slice is W cos α; one equation (moment equilibrium of entire mass), one unknown (factor of safety)
Bishop's modified method (Bishop 1955)	Accurate method; only for circular slip surfaces; satisfies vertical equilibrium and overall moment equilibrium; assumes side forces on slices are horizontal; N+1 equations and unknowns
Force equilibrium method	Satisfy force equilibrium; applicable to any shape of slip surface; assume side force inclinations, which may be the same for all slices or may vary from slice to slice; small side force inclination result in values of F less than calculated using methods that satisfy all conditions of equilibrium ;large inclination result in values of F higher than calculated methods that satisfy all conditions of equilibrium; 2N equations and unknowns
Janbu's simplified method (Janbu 1968)	Force equilibrium method; applicable to any shape of slip surface; assumes side force are horizontal (same for all slices);factors of safety are usually considerably lower than calculated using methods that satisfy all conditions of equilibrium ;2N equations and unknowns

Modified Swedish method (U.S. Corps of Engineers 1970)	Force equilibrium method; applicable to any shape of slip surface; assume side force inclinations are equal to the inclination of the slope (same for all slices);factors of safety are often considerably higher than calculated using methods that satisfy all conditions of equilibrium; 2N equations and unknowns
Lowe and Karafiath's method(Low &Karafiath1960)	Generally most accurate of the Force equilibrium methods; applicable to any shape of slip surface; assumes side force inclinations are average of slope surface and slip surface(varying from slice to slice)satisfies vertical and horizontal force equilibrium; 2N equations and unknowns
Janbu's procedure of slices (Janbu 1968)	Satisfies all conditions of equilibrium; applicable to any shape of slip surface; assumes heights of side forces above base of slice (varying from slice to slice);more frequent numerical convergence problems than some other methods;accurate method; 3N equations and unknowns
Spencer's method (Spencer 1967)	Satisfies all conditions of equilibrium; applicable to any shape of slip surface; assumes that inclinations of side forces are the same of every slice; side force inclination is calculated in the process of solution so that all conditions of equilibrium are satisfied; accurate method; 3N equations and unknowns
Morgenstern and Price's method (Morgenstern & Price 1965)	Satisfies all conditions of equilibrium; applicable to any shape of slip surface; assumes that inclinations of side forces follow a prescribed pattern, called f(x); side force inclinations can be the same or can vary from slice to slice; side force inclinations are calculated in the process of solution so that all conditions of equilibrium are satisfied; accurate method; 3N equations and unknowns

Sarma's method (Sarma 1973)	Satisfies all conditions of equilibrium; applicable to any shape of slip surface; assumes that magnitudes of vertical side forces follow prescribed patterns; calculates horizontal acceleration for barely stable equilibrium; by prefactoring strengths and iterating to find the value of the prefecture that results in zero horizontal acceleration for barely stable equilibrium, the value of conventional factor of safety can be determined; 3N equations & unknowns

Table 2-1 (continued)

For the purpose of this study, we will adopt the Bishop's method of slices. This may be one of the most widely used methods of slope stability analysis. In this method, the soil located above the probable surface of sliding is divided into vertical slices as shown in fig. 2-2. Considering unit length perpendicular to the cross section, the forces that act on a typical slice are as shown in fig. 2-3.

Assuming that;

$E_1 = E_2$ And $T_1 = T_2$

The following formula was arrived and applied on wide scale.

$$F = \frac{\sum \left[c\, b + (W - u\, b) \tan \phi \right] 1/ m_\alpha}{\sum W \sin \alpha}$$

Where;

F = Factor of Safety,

$m_\alpha = \cos\alpha \; [\; 1 + (\tan\alpha \tan\phi)/ F \;]$,

c = cohesion intercept,

ϕ = angle of internal friction of soil,

W = weight of individual slice (γbd),

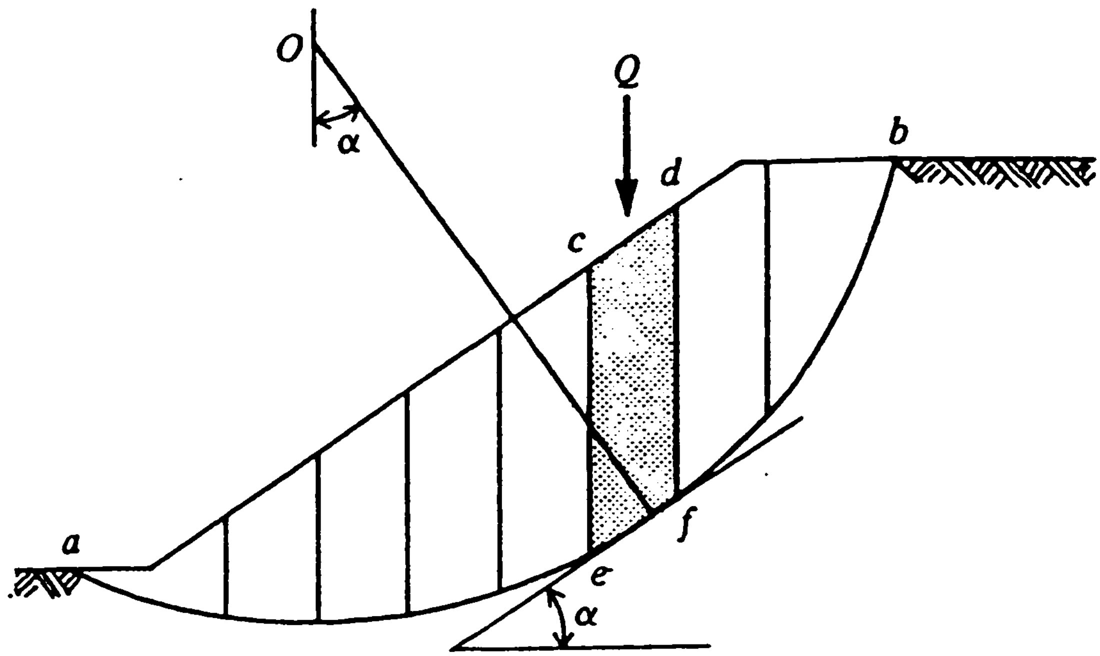


Fig. 2-2. Division of slope into slices

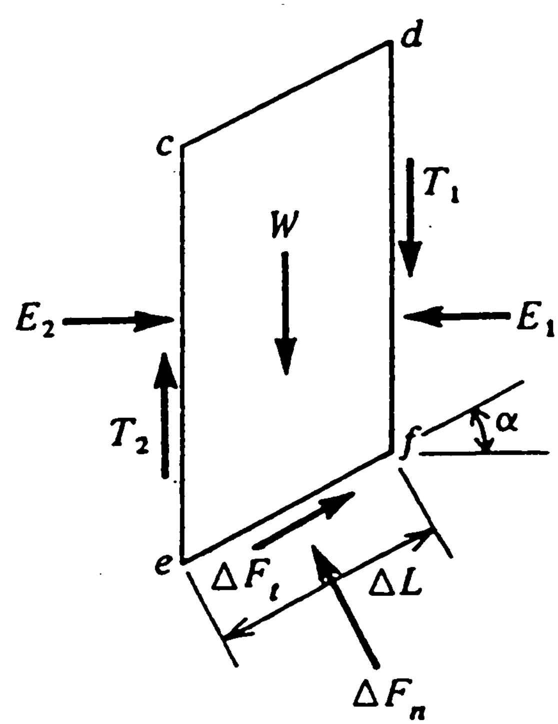


Fig. 2-3. Forces on a slice

b = width of slice,

u = pore water pressure,

α = angle between base of slice and horizontal.

From the above formula, it can be concluded that the main parameters that affect the slope factor of safety are:

1- Soil cohesion intercept (c),

2- Angle of internal friction(ϕ),

3- Pore water pressure (u).

4- Slope height,

5- Slope inclination,

Chapter: 3

LANDSLIDE CAUSES AND MITIGATION

As mentioned in the previous chapter, the Factor of Safety is basically a ratio between shear strength and shear stress. Therefore, it is affected by any factor that contributes to high shear stress or to low shear strength. These factors can be caused by nature, human activities or combination of both. Hereunder we will try to classify the major factors that contribute to an increase in the possibility of landslide.

3.1 NATURE RELATED FACTORS:

3.1.1 Erosion

Erosion by a stream or a river to natural slope causes a removal of lateral or underlying support. Removal of lateral or underlying support increases the shear stress on the slope. Accordingly, this will reduce the Factor of safety and increases the possibility of landslide. Erosion was a trigger for 80% of earth flow in the province of Quebec in the period of 1840-1980 (Rissmann et al., 1985).

3.1.2 Earthquakes

Over the years, earthquakes have triggered many landslides. Prime example is the landslide of Peru 1970 where a landslides, triggered by an earthquake, have caused death to 20,000 people. Earthquakes create transitory earth stress. Their action on slope is complex, involving an increase in shear stress and in the same time, a decrease in shear strength (Varnes, 1978).

3.1.3 Rainfall

Rainfall contributes to increase in pore water pressure. Intergranular effective pressure due to capillary tension in moist soil is destroyed upon saturation (Varnes, 1978). That is a decrease to soil shear strength. On the other hand, rainfall weight can act as extra surcharge on a slope. That is an increase in shear stress.

3.1.4 Snow

Similar to rainfall, snow weight act as surcharge on slopes and accordingly increase shear stress. Also, snow melt contributes to increase of water pressure and a decrease in shear strength.

3.1.5 Temperature change

Freezing of water in cracks introduces a lateral pressure. This is increase in shear stress. As mentioned in (3.1.4) above, snow melting contributes to decrease in shear strength. On the other hand, drying of clays results in cracks and loss of cohesion and allows water to seep in.

3.2 HUMAN RELATED FACTORS:

3.2.1 Surcharge:

Any surcharge contribute to an increase in shear stress and accordingly decrease of the factor of safety. The surcharge from human agencies may be weight of buildings and other structures, fills, waste piles, or weight of water from leaking pipelines.

3.2.2 Removal of lateral support:

Removal of lateral support increases shear stress on the slope. Human activities which may cause removal of lateral support include:-

a- cuts for roads and canals.

b- removal of retaining walls and sheet piling.

3.2.3 Removal of underlying support:

Removal of underlying support contributes to an increase in shear stress. Prime example for this case is excavation for mining activities.

3.2.4 Deplanting

By removal of trees and vegetation, we remove a reinforcing agent which contributes to an increase in soil shearing strength. In addition, clearing of vegetation causes a changes to intergranual forces due to water content and pressure in pores. Such changes contribute to a reduced shear strength (Varnes, 1978).

3.2.5 Ground water movement

Fluctuation of ground water table has a negative effect to soil strength. Human activities which cause such movement include diversion of streams and rapid drawdown by removal of water from canals or wells. Rapid change of water table elevation produces spontaneous increase of pore water pressure (Flawn, 1970).

3.3 LANDSLIDE MITIGATION

Landslide hazard can be reduced by many ways. Restriction of development in landslide prone areas may be the most important trend. However, making landslide zones as human free land is not always possible. In this case, other methods can be applied to reduce landslide possibility. These methods include drainage system, slope grading and soil stabilization. Schuster (1996) has shown that mitigation measurements could reduce landslide losses by more than 90%. The following are some examples of slope correcting methods.

3.3.1 Drainage system (surface and underground)

One of the most important ways to increase the stability of slope is through drainage system. Surface and groundwater drainage has shown success and cost effectiveness in slope stabilization. Drainage systems include drainage galleries, drain holes and drainage blankets.

Drainage will reduce the weight of mass tending to cause the landslide. On the other hand, it will increase the strength of the materials in the slope. Thus, it helps reducing the stress and increasing the resistance.

3.3.2 Slope grading

The stability of slopes can be increased by improvement of slope geometry. Slope may be graded by removal of the soil at the top of the slope or by placing pressure at the bottom. Grading of the slope reduces the driving force toward slope failure and increase the factor of safety.

3.3.3 Lateral support

Lateral support is another method to increase the slope stability. This can be through buttresses, counterweight fills, or retaining walls. Lateral support will provide additional dead load to reduce slope movement.

3.3.4 Soil strengthening

Soil strengthening can be very effective In reducing landslides. Strengthening may be through soil stabilization, piling, earth reinforcement or planting. All these methods will contribute to increase of resistance to slope failure.

3.4 LANDSLIDE STABILIZATION CASE HISTORY

The following is a case study for the application of mitigation measurements to a landslides. It is presented here as a good example for the application of various slope stabilization methods. The study has also included back analysis calculations for estimating in place shear strength and field performance evaluation of a completed project. These approaches may be used for reducing risk in our landslide risk assessment case study at Repentigny city (Chapter 7).

The study was prepared by Grefsheim et al. (1984). The site is an area of noted unstable slopes because of the generally hilly terrain and presence of overconsolidated clay subsoils. Studies have been conducted for other nearby sites for evaluating in-place shear strength of the clay subsoils.

Studies of in-place shear strength have consisted of back-analysis calculations for failed slopes, laboratory shear strength testing, and identification tests for use in correlations. From back-analysis calculations at four existing landslides, including the subject case study, values of in-place shear strength of the clay subsoil plotted against average plasticity index have been presented. Using this data and other results of other studies of similar soil, an estimated shear strength factor u_r was plotted over a range of plasticity index of 20 to 80. For an assumed effective cohesion, the strength factor u_r is equal to tan ϕ', where ϕ' is the effective internal friction angle.

For this case study, the estimated critical failure surface is mostly in the clay subsoil with average plasticity index of 58. Using the correlation cited, the resulting effective internal friction angle is $\phi' = 13°$. This shear strength value was utilized to aid in establishing the estimated failure surface and depth to stable subsoils (Grefsheim 1984).

The potentially unstable soils extended an estimated 20 feet below desired finished grades, and use of permanent earth tiebacks was necessary as part of the retaining wall design. The general method of a driven pile system with tiebacks had been applied on other nearby projects both for initial grading design and for stabilizing failed slopes.

3.4.1 Site Geology

The site is just east of the fall line in the region of the middle Atlantic coastal plain deposits near the east coast of the United States. The natural geologic profile generally consists of Pleistocene age stream terrace and colluvial deposits at the ground surface overlying the overconsolidated

Cretaceous age sediments. The colluvium typically occurs as a fairly thin mantle along hillslides. This soil layer is the result of erosion and gravity transport downslope.. Consisting of a non-uniform mix of sand, clay and gravel, the colluvium or stream terrace deposits form a source of perched water flow especially below fill. The stream terrace is relatively coarse, mostly sand and gravel with boulders, and is a competent soil with high shear strength compared to the more recent colluvium.

The underlying Cretaceous age sediments generally include intermittent layers of fine sand, silt and clay with gravel at some locations. A very stiff or hard clay layer with blocky structure is often the predominant subsoil as for this case study. This soil deposit which is part of the Potomac Group sediments of this area, is known to be overconsolidated at least about 12 to 15 tons per square foot in excess of the existing overburden pressures. As overburden stresses are relieved, the clay soil tends to expand. In the upper 10 to 20 feet below the ground surface, more intense weathering causes the blocky structure. Vertical and subhorizontal breaks can develop into a continuous failure surface. Slickensides, which are often noted in soil samples, are evidence of previous slides occurring in this subsoil. An effective soil cohesion of near zero and low residual friction angle must be used for grading design involving this overconsolidated clay.

3.4.2 Site and Landslide Description

The site comprises the sloping rear yards of two residence along the bluffs overlooking the tidal flats of the Potomac River south of Washington, D.C. Underground utility lines on the site consist of a main gas feeder line and both a sanitary and a storm sewer line. Figure 3-1 shows landslide limits and surrounding structures. A cross section through the slide is also shown which indicates the moderate slope involved and the estimated failure surface considered for design of the remedial construction.

The original development of the two affected residences included placing about 10 feet of fill along an existing

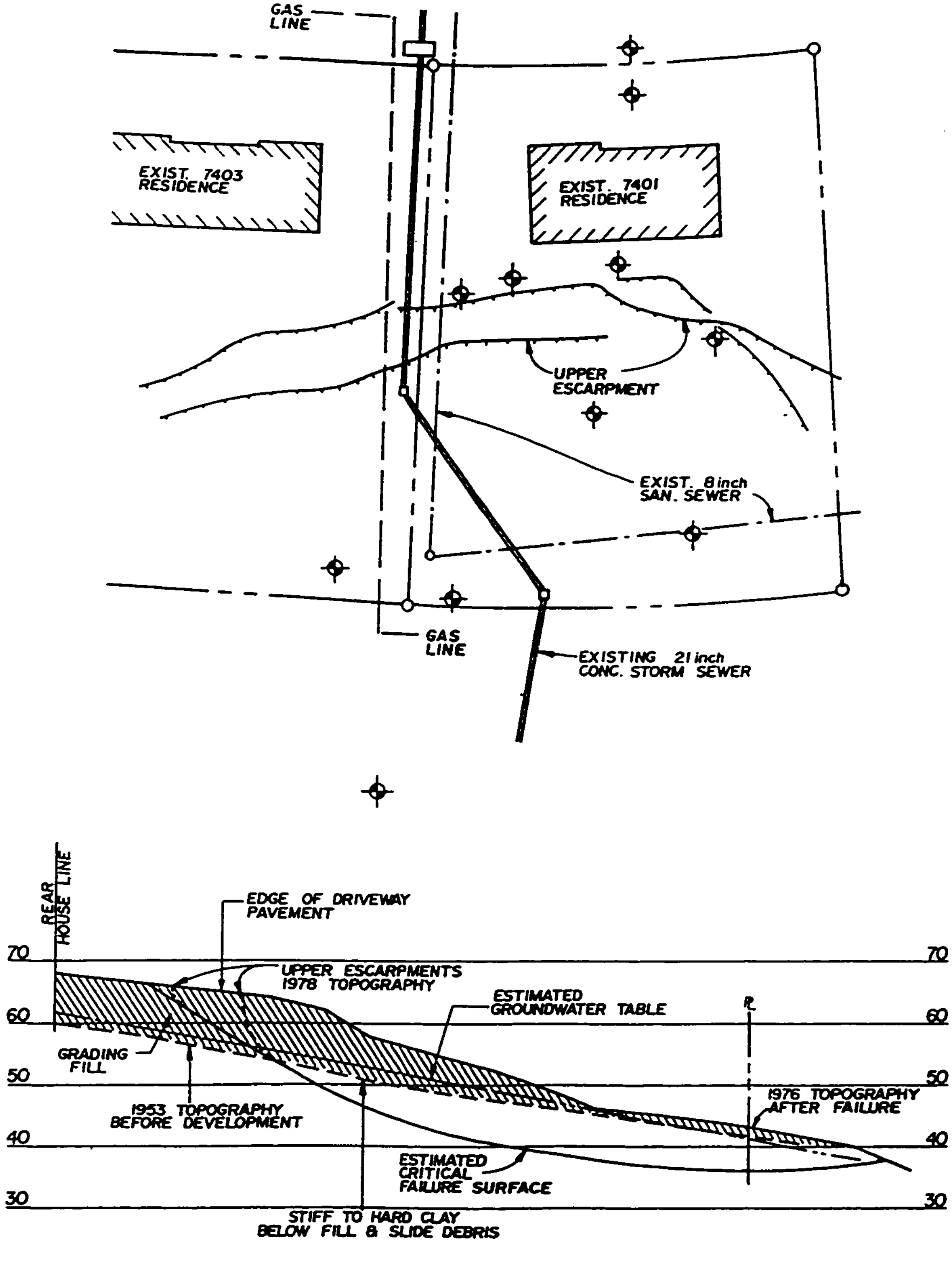


Figure 3-1 Plan and Cross Section of Landslide
(After Grefsheim 1984)

natural slope of about 5H: 1V. The finished slope was a maximum of 3H: 1V prior to the initial slope movements which were noted in 1970 about ten years after the original construction. Movements along the slope occurred slowly and sporadically from about 1970 to 1978 and these were recorded by topographic surveys taken in 1970, 1976 and 1978.

Various grading and landscaping reparations were made as these relatively minor slope movements occurred. This included some resurfacing of cracked paving at the top of slope, minor filling also at the top of the slope; and excavation beyond the toe of the slope to remove slide debris. By early 1978 the storm sewer line was known to be broken and its leakage was contributing to movements.

There were obvious liability considerations and a hazard because of the existing gas line along the failed slope. More extensive studies were therefore undertaken in order to develop a reliable stabilization plan.

3.4.3 Field Investigations, Subsoils and Slope Analysis

Field investigations, including four soil test borings, had been made during 1970 through 1976 for relatively limited evaluations of the slope movements during that period. The study began in 1978 included seven more test borings and four slope inclinometers.

The plan of Figure 3-1 indicates the test borings and slope inclinometer locations. The general soil profile, consisting of grading fill, disturbed soils to the depth of slope movements, and the very stiff to hard clay subsoil, are shown by the cross section of Figure 3-1.

The slope movements were probably initiated by filling the top of a slope which may already have been at an incipient failure condition. Despite the relatively shallow original slope, of 5H: 1V, spring conditions noted throughout the general area would result in a very low effective shear resistance due to the high natural pressures that tend to develop. Besides grading to finished slope of 3H: 1V over a 20 to 30 foot slope height, it is likely that flow from natural springs was blocked as a result of the original site development. The initial slope repairs, which included filling at the top and debris excavations at the base of the slope, contributed to the movements. The eventual breakage of a 21 inch storm sewer line accelerated movements along the slope.

The hard clay subsoil is from the local Potomac group deposits and is typical of the highly overconsolidated clay of the general area. For the upper weather zone, the cohesion is near zero and effective shear strength is given by the internal friction angle, $\phi' = 13°$ as estimated for this site. Slope inclinometer data at 3 locations along the failed slope indicated a failure surface extending about 7 feet into the overconsolidated clay which is within the typical upper weathered zone of this soil stratum. The estimated failure surface shown in Figure 3-1 is a best fit for the slope inclinometer readings and the survey data which show the upper escarpment and uplift zone.

Slope stability calculations were based on a slope profile determined from the 1970 topography taken after limited slope movements consisting of horizontal opening of cracks in the paving along the eventual upper escarpment. The ground water seepage line was taken at the base of the fill or just above the clay layer. This fits with readings taken in the test borings and also reasonable for a perched water table in this soil profile.

Placement of the fill for the original site grading apparently initiated the slope failure which developed over the next several years. In addition to the added weight on top of the slope, higher water pressures could be developed in the underlying clay natural soil. Continuous flow paths from percolation of surface runoff would tend to develop in the clay fill and extend into the blocky natural clay of the upper weathered zone. No longer exposed to drying, continuous water columns would tend to develop in which hydrostatic pressures would be very high. With continuous exposure to this water condition, a system of relatively short and discontinuous breaks within an already blocky soil mass would tend to develop and eventually from a continuous failure zone.

The groundwater table shown in Figure 3-1 should accurately model the effective ground water seepage conditions for use in the slope stability analysis. The Potomac clay subsoil comprises most of the failure surface, and this slope failure provides data for back analysis to evaluate in-place shear strength of this clay. Variation of shear strength for the overlying fill do not significantly affect the factor of safety for the slope stability. The following estimated soil or slope properties were utilized:

Effective Soil Friction, Fill $\phi = 25°$

Effective Soil Cohesion, Fill $c' = 0$ lbf/ft

Wet unit weight, Fill $\gamma_w = 125$ lbf/ft^3

Effective Soil Cohesion, Clay $c' = 0$

Wet unit weight, Clay $\gamma_w = 120$ lbf/ft^3

Groundwater Table, as shown on Figure 3-1

Failure Surface, as shown on Figure 3-1

A required effective clay soil friction, $\phi' = 14.9°$ is found in the back analysis for a factor of safety, f.s. = 1.0. An average plasticity index along the failure surface has been estimated at 58 and this would indicate an effective friction angle, $\phi' = 13°$ based on the P.I. correlation cited earlier.

3.4.4 Stabilization Design and Related Construction

Figure 3-2 indicates the remedial construction completed on this project consisting of slope stabilization, isolation and re-routing of utility lines, and a new storm sewer line. The slope stabilization consisted of retaining structures and subdrainage. The retaining structures permitted developing a stable terrace and driveway area along the rear of the houses, and grading the remaining lower slopes to a flatter stable condition.

Typical retaining wall sections are also shown by Figure 3-2 consisting of H-Beams, pressure-grouted earth tiebacks, and water connections. Details of a typical segment along the upper wall are as follows:

HP 14x73 Pile sections, 4 feet on-center, 36 feet Pile length,

Pressure grouted earth anchors 4-feet on-center,

38 to 40 feet anchor length, 94 kips design loads.

Piles for the upper wall were driven to about 25 feet below the estimated depth of failure surface.

Design for stabilization construction was accomplished before complete slope inclinometer data was available and it was necessary to utilize conservative estimates for depth of disturbance and lateral active pressures. Figure 3-3 illustrates the essential design assumption with comparison to the estimated slope failure conditions. Full lateral active pressure was assumed acting from the top of wall at El 66 down to a design "dredge line" at El 42. An equivalent fluid pressure of 90 pcf was utilized for lateral active pressure to approximate loading of the unstable soil mass upslope. Also shown on Figure 3-3 is a force diagram considering the unstable soil mass as a sliding wedge along a failure surface with zero friction resistance.

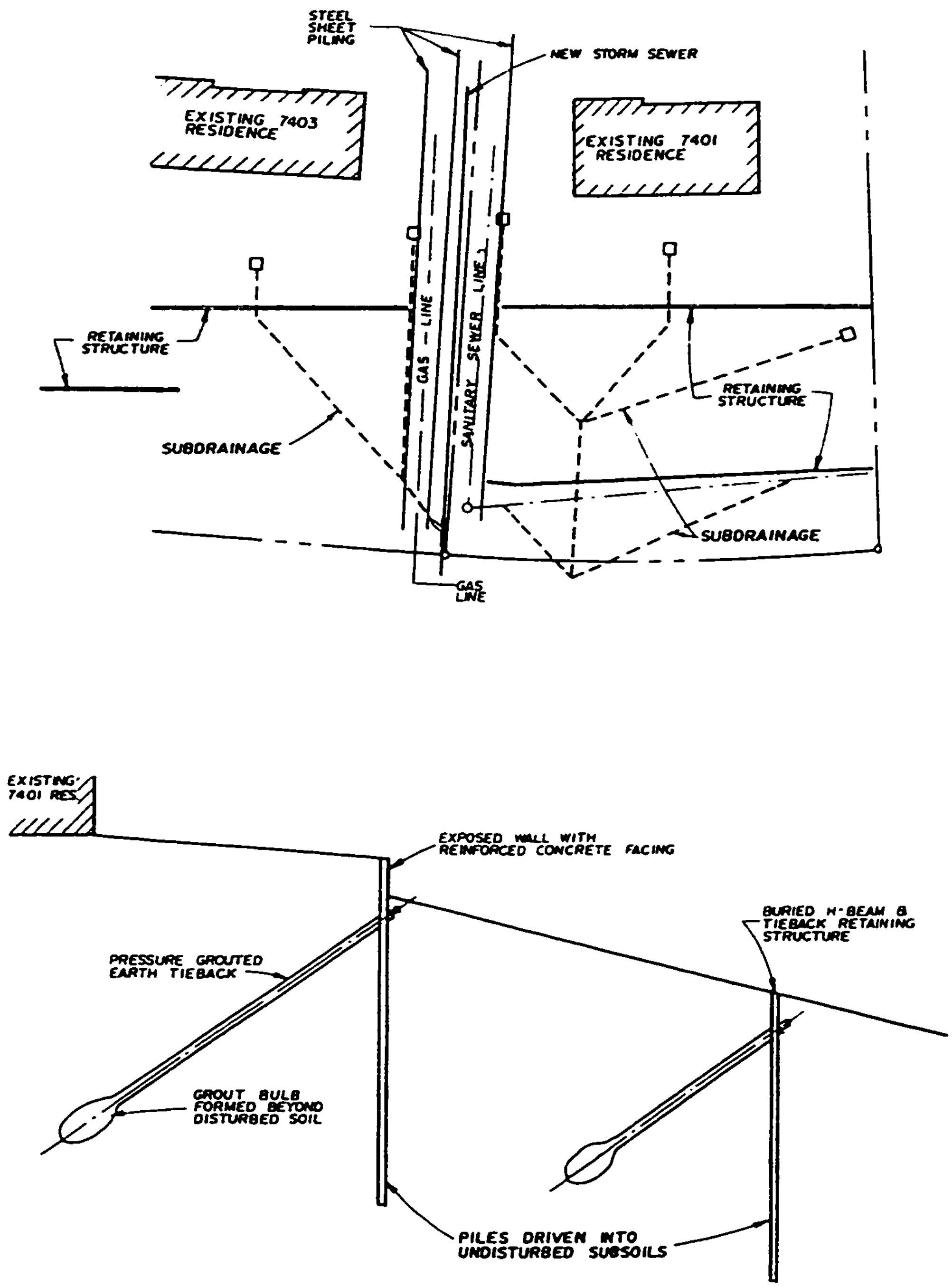


Figure 3-2 Remedial construction

(After Grefsheim 1984)

The resulting horizontal thrust of 24.5 kips per foot along the wall line is approximately equal to the equivalent concentrated load based on the design equivalent fluid pressure of 90 pcf. As shown by the schematic diagram of Figure 3-3, the unstable soil mass is assumed to extend upslope to the house line. Before completion of the slope stabilization construction on this project, the upper escarpment had propagated upslope approximately as shown by the assumed failure wedge of Figure 3-3.

The regions between and below the lines of retaining structure were stabilized by installing subdrainage, and grading to a finished slope 4H: 1V. There was no attempt to break up the established failure surface, and the possibility of relatively limited future movements of this portion of the slope was accepted. The subdrainage installations were included in this portion of the design as a minimum stabilization procedure. The layout of the shallow subdrainage is shown by Figure 3-2. A typical section of this subdrainage is shown by Figure 3-4. Seepage or spring conditions due to perched water was noted

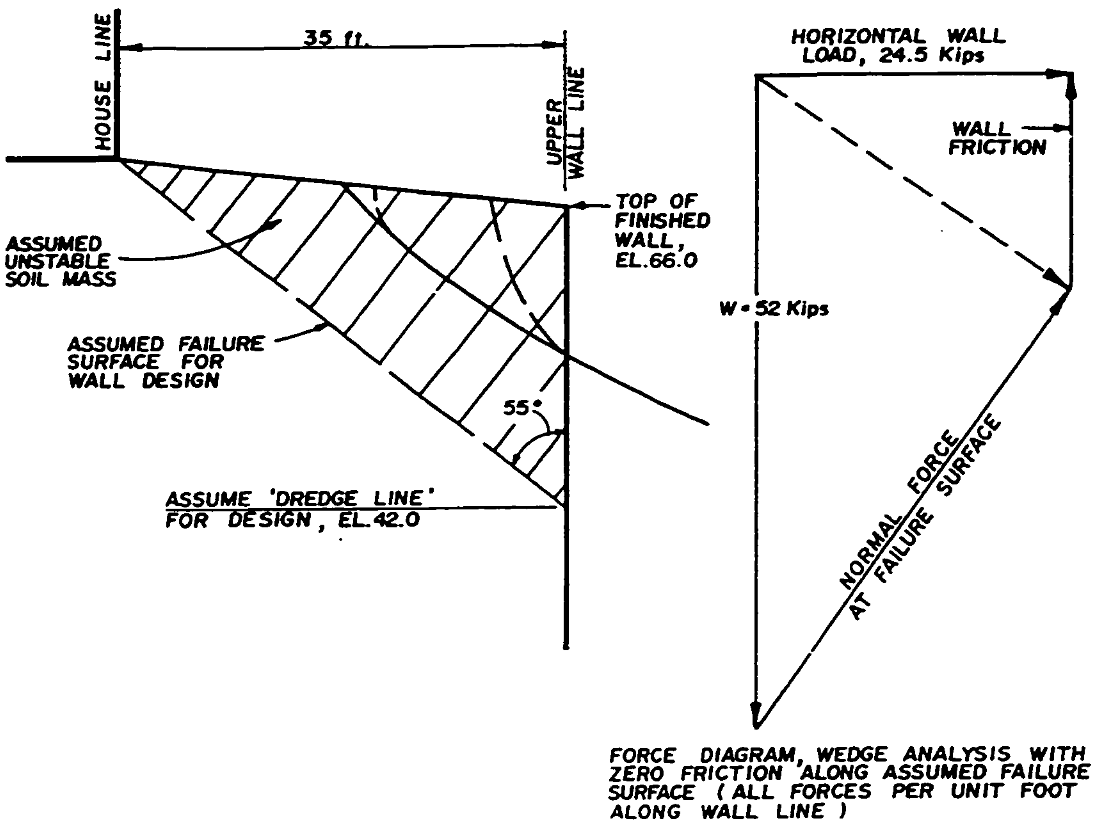


FIGURE 3-3 LATERAL LOADING

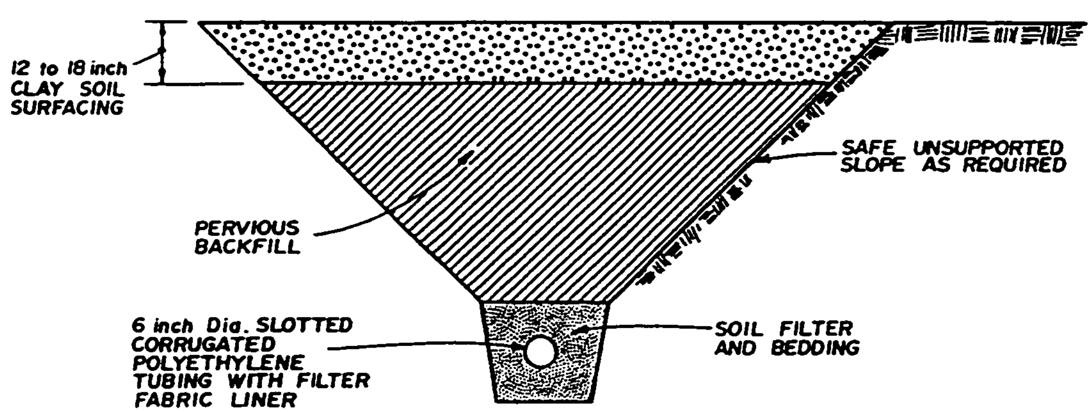


FIGURE 3-4 SUBDRAINAGE SECION

within much of these lower portions of the slope. Based on these observations and the mixed clay and sand subsoils, control of seepage was expected to contribute significantly to limiting future shallow disturbances.

The retaining walls and subdrainage could be expected to provide a safe finished slope except that minor disturbances could still be anticipated for relatively unprotected zones beyond the retaining walls. For this reason, additional stabilization construction was necessary for protection of utility lines. Separate sheeting systems, consisting of four lines of steel sheet piling, were installed along the slope as shown on Figure 3-2. These protected the gas line, a new storm sewer line and the sanitary sewer. As an added precaution, the gas line, which was originally along the base of the disturbed slope, was extended across the highway below the slope and then a new line parallel to the slope well beyond the region of instability.

3.4.5 Construction Observations, Testing and Design Adjustments

Routine observations and testing during construction included monitoring of the H-Pile and sheet pile installations, testing of tiebacks and checking of the subdrainage. Construction for this project involved conflicts among the several subsurface structures to be installed and also between these proposed installations and the existing deep foundations of the residence upslope.

Adequate pile depths were readily checked to assure a minimum required penetration into the undisturbed subsoils. The earth tiebacks were tested by loading in specified increments to 1.5 times the design load. Each tieback was then locked off at 0.75 times the design load. This proof resting was conducted on selected tiebacks.

The tieback installation and testing preceded the subdrainage installation, but this preferred sequence was not feasible where the subdrainage line crossed the walls. Portions of the subdrainage construction were therefore completed before the retaining walls, so backfilling could be completed before the tieback testing and lock-off. This was necessary to provide and maintain the reaction against the tieback load. The original subdrainage layout was revised in order to limit this conflict. The final layout as shown by Figure 3-2 includes crossing at three locations.

Another potential conflict consisted of the existing deep foundations of the residence upslope. The tiebacks as designed required pressure grouted bulb which extended about 10 feet beyond the rear house line and would have to clear six existing drilled piers along the rear bearing wall. Location and depth of the piers could not be established with complete assurance.

However, reasonable assumptions indicated the tiebacks would be safely below the pier bases. A plan of increased tie angle was considered to avoid these existing foundations but additional ties would be necessary because of the increased horizontal component. A decision was made to install according to the original design considering that actual conflicts were unlikely and also that adjustments could be done safely if existing foundation piers were encountered during drilling of the tiebacks. This procedure was successful and there was no incidence of the obstruction or need to adjust the tieback design.

3.4.6 Post Construction Observations

Readings at survey stations and one slope inclinometer were made during a three year period after the slope stabilization construction. Visual inspections were also made during that initial post-construction period and in following years. Locations of the observation stations are shown in figure 3-5.

Readings taken at the five survey stations placed in the upper wall consisted of horizontal alignment. Survey readings reported to the 0.0001 ft accuracy initially showed apparently significant movements during the summer months just after completion of the construction. Deflections up to 6 inches

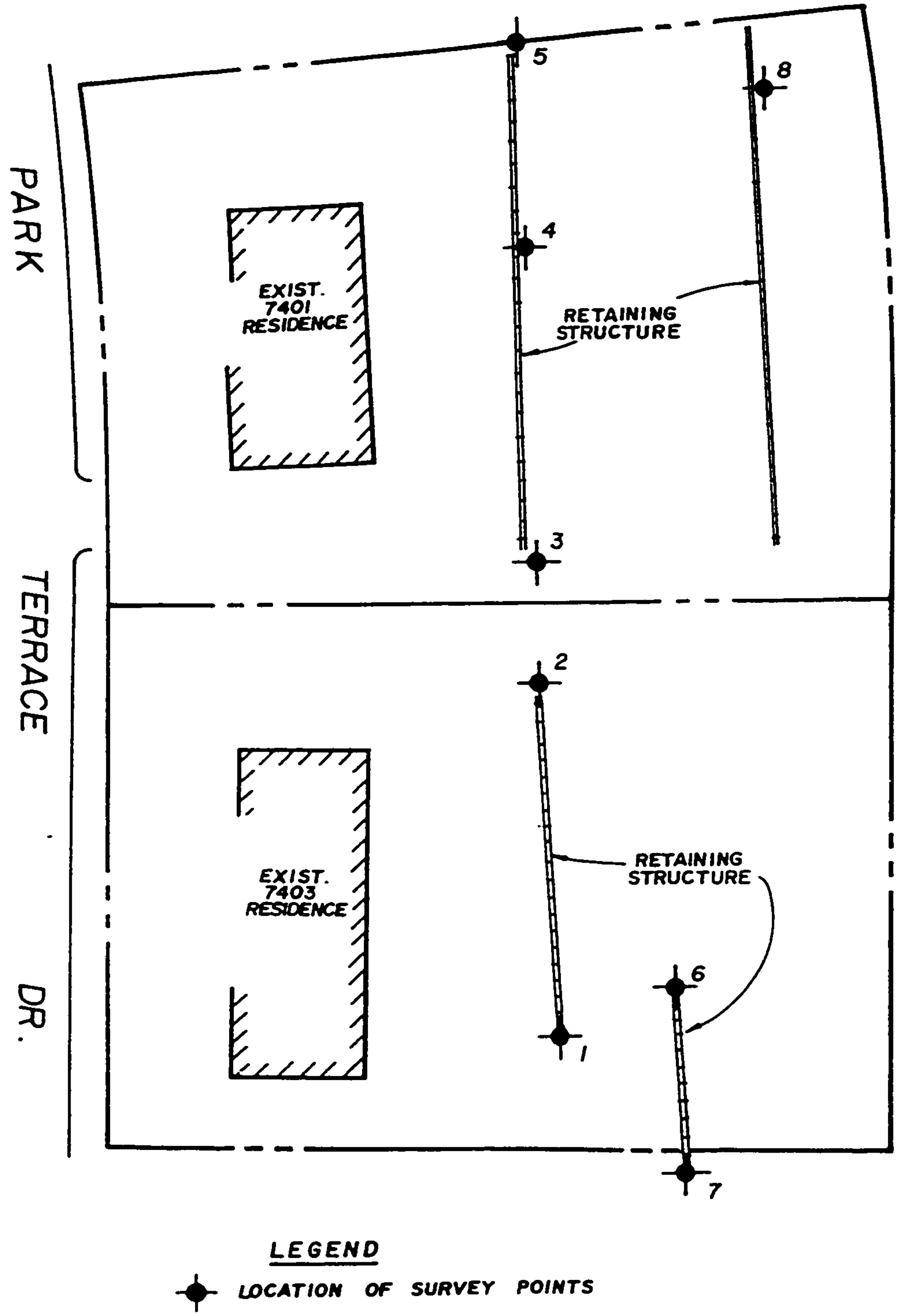


FIGURE 3-5 POST CONSTRUCTION OBSERVATION STATIONS

initially reported were found to reverse in later survey. A maximum apparent net deflection of about 1 inch was reported in the readings which were taken about every 2 months for a period of 14 months.

Smaller apparent deflections of less than ½ inch were recorded at the three survey stations set in the lower and intermediate walls. Small cracking, less than 1/32 inch opening, was noted in the reinforced concrete facing along the exposed portion of the upper wall.

Based on the survey readings of the wall alignment, including the minor cracking, it could be concluded that the installation was stable. The apparent larger deflections noted in initial surveying were attributed to temperature variations and/or survey error.

Slope inclinometer readings were taken at a casing re-installed near the lower wall line. Original inclinometer casings were destroyed by the construction. An apparent small deflection of less than 1/4 inch was recorded in the upper 5 to 10 feet. This was attributed to probable disturbance of this casing rather than an indication of shallow slope movements. The inclinometer readings did not show any deep seated movements.

Chapter: 4

RISK AND RELIABILITY ANALYSIS IN SLOPE STABILITY

Reliability of engineering systems designed to achieve desired objectives and avoid failure has long been a technical interest as well as a public concern. Throughout the years, methods have been developed for the piecemeal reliability evaluation of some aspects of specific components of an engineering system. A typical example is frequency analysis which deals with the uncertainty associated with natural randomness in hydrology, wind, and seismic engineering.

Methods of hazard estimation range from reliability analysis to purely empirical estimates. That is normally expressed as a failure probability. Most methods tend to provide a hazard evaluation under conditions of uncertainty. The uncertainty most frequently dealt with in engineering systems includes future geophysical events such as rainfall, floods and earthquakes.

4.1 UNCERTAINTY IN SLOPE ANALYSIS

The assessment of slopes and landslides is difficult because of many uncertainties. The variability of earth materials over a site leads to imperfection of the knowledge about shear strength. Pore water pressure may vary widely over a site. Even for the same location pore pressure may change with time. Minor geological details may remain undetected and thus the actual mode of failure may be different from the one assumed in the analysis.

Consequently, the shear strength may vary considerably even within so-called homogeneous soil deposits. Decades before the recent development of probabilistic geomechanics, Bishop (1948) reported variations in the undrained shear strength of London clay. The results were summarized as shown in table 4-1. Coefficients of variation ranged from 18% to 42%. The mean shear strength varied by a factor of 6 from 16 p.s.i. to 99 p.s.i.

Depth Feet	Samples from Whole Site		Samples from Pit 218 only	
	No. Of Samples	Coefficient of variation	No. Of Samples	Coefficient of variation
0-1	38	25%	19	20%
1-2	49	18%	17	26%
2.4	77	38%	19	42%
4-8	65	30%	13	35%
8-12	86	36%	-	-
12-16	46	35%	-	-
16-20	32	29%	-	-
23-27	12	30%	-	-
30-60	43	42%	-	-
75-105	23	33%	-	-

TABLE 4-1

Coefficient of Variation of Undrained Shear Strength of London Clay Reported by Bishop (1948)

4.2 THE PROBABILISTIC APPROACH

The probabilistic approach is the most used one in hazard estimation methods. It has the advantage that it allows for a logical and systematic analysis of uncertainty. Computer analysis and mathematical models are normally required.

A probabilistic approach recognizes that any earth structure has some probability of failure however small. In contrast, a deterministic approach leads to an expectation that failure of a slope cannot occur under the conditions for which the calculated Factor of Safety is greater than one. Yet failures of slopes designated to be safe (i.e., calculated $F > 1$) are not uncommon.

4.2.1 Safety Factor and Safety margin as Random Variables

Each parameter affecting slope stability may be regarded as a random variable, with an associated probability distribution, rather than a constant. Variability of some parameters (e.g., unit wt. and geometrical parameters) has an insignificant influence in slope stability.

Such parameters may be regarded as constants whereas it is desirable to consider shear strength parameters and pore water pressure as random variables. Thus the factor of safety F which may be defined as the ratio of resisting forces or moments RF and disturbing forces or moments DF is itself a random variable. Similarly, the safety margin SM which may be defined as the difference of RF and DF is also a random variable (Chowdhury 1984).

4.2.2 Reliability and Failure Probability

Reliability is the probability that a slope will be safe. That means that the slope will survive and not fail under given conditions. It is therefore the probability of success p_s which is the probability that the factor of safety F will be greater than one or that the safety margin SM will be greater than zero, i.e.,

$$p_s = P\ [F > 1] = P\ [SM > 0] \qquad (1)$$

The probability of failure P_f is complementary to the probability of success and is defined as:

$$p_f = P\ [F \leq 1] = P\ [SM \leq 0] \qquad (2)$$

It is axiomatic that

$$p_s + p_f = 1 \quad (3)$$

Provided that the probability distribution of either F or SM is known, the reliability p_s can be calculated as the area under the probability distribution curve to the right of the ordinate at F =1 or SM = 0. Similarly, the probability of failure is the area to the left of the ordinate as shown in Fig.4-1 for two well-known distributions, the normal or Gaussian distribution and the lognormal distribution.

4.3 METHODS OF PROBABILISTIC CALCULATIONS

The factor of safety of slope F is a function of several random variables such as the soil cohesion c, angle of internal friction ϕ, unit weight γ, slope inclination α, slope height H, pore water pressure u etc. Some or all of these parameters may be regarded as stochastic parameters, each with its own probability distribution. The probability distribution of F may be determined from the assumed probability

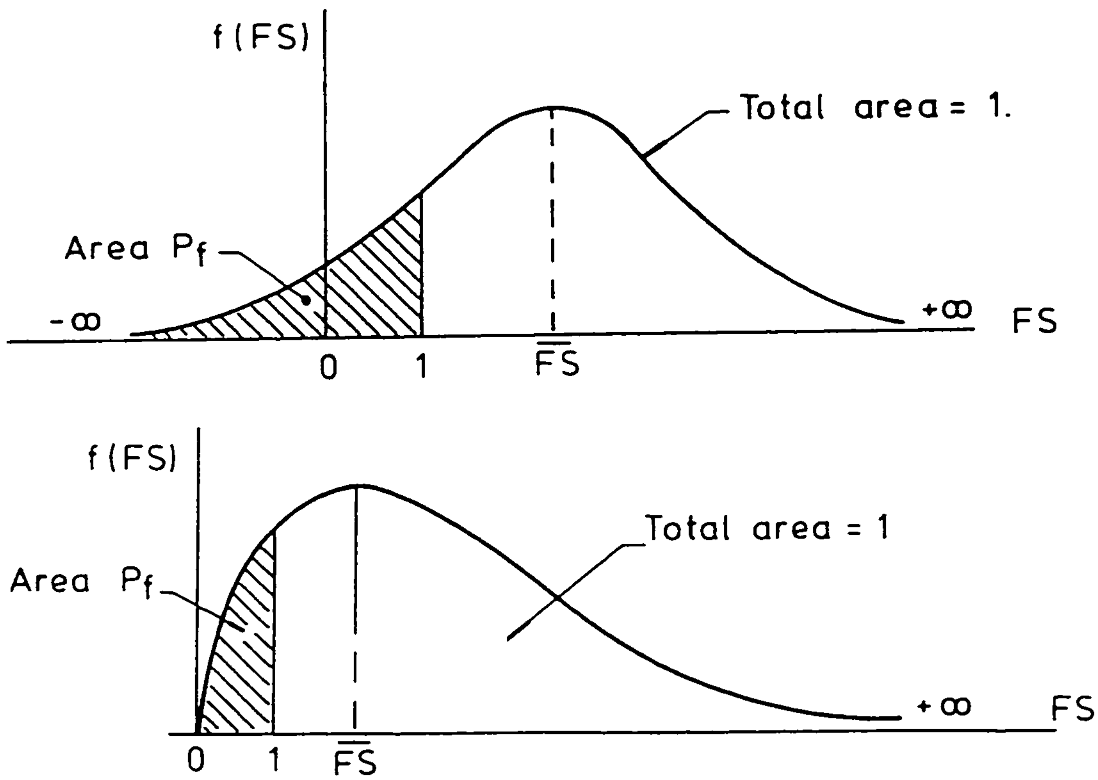


Fig. 4-1 Two probability distributions of the factor of safety FS, frequently used for probabilistic calculations in geomechanics. (After Chowdhury 1984)

distributions of the basic stochastic variables.

One method which has been commonly used by geotechnical engineers to do this is the Monte Carlo simulation technique. The use of computer is almost essential because the calculations are repetitive and time consuming. Alternatively, one may assume a probability distribution for F; commonly used assumptions are a normal or Gaussian distribution, log-normal distribution and a beta distribution.

Even if the shape of distribution is assumed, the statistical parameters of the distribution must be estimated or derived from the statistical parameters of the basic variables. Two approximate methods are widely used to calculate the statistical parameters of F, namely the Taylor series approximation and Rosenblueth's method. (Chowdhury 1984).

4.3.1 Monte Carlo Simulation Technique

The Monte Carlo simulation method enables the determination of the distribution as well as the estimation of the statistical moments. Neither the Taylor series approximation nor Rosenblueth's methods are required if simulation is the approach adopted. The Monte Carlo simulation technique involves the generation of random numbers and a value of F associated with a set of random values of the basic stochastic variables is simulated.

After many values of F have been calculated in this way, the probability distribution of F is generated and the statistical parameters may be calculated. The probability of failure may be estimated from the generated distribution or directly by calculating the relative frequency with which F was found to be less than or equal to one during the simulation.

4.3.2 Taylor Series Method

The Taylor series method is a first order, second moment approximation. Given the mean and standard deviation of each of the basic variables, the mean and standard deviation of F is calculated. Higher moments of the basic variables could be used for greater accuracy but this is generally not done in order to retain the basic convenience and simplicity of the approach.

Moreover, data on higher statistical moments of soil parameters is lacking. This method has proved to be successful in most slope stability problems (Chowdhury 1984). Derivatives of the function F are required and the method is unsuccessful or inapplicable for those geotechnical problems in which one or more derivatives of F cannot be found.

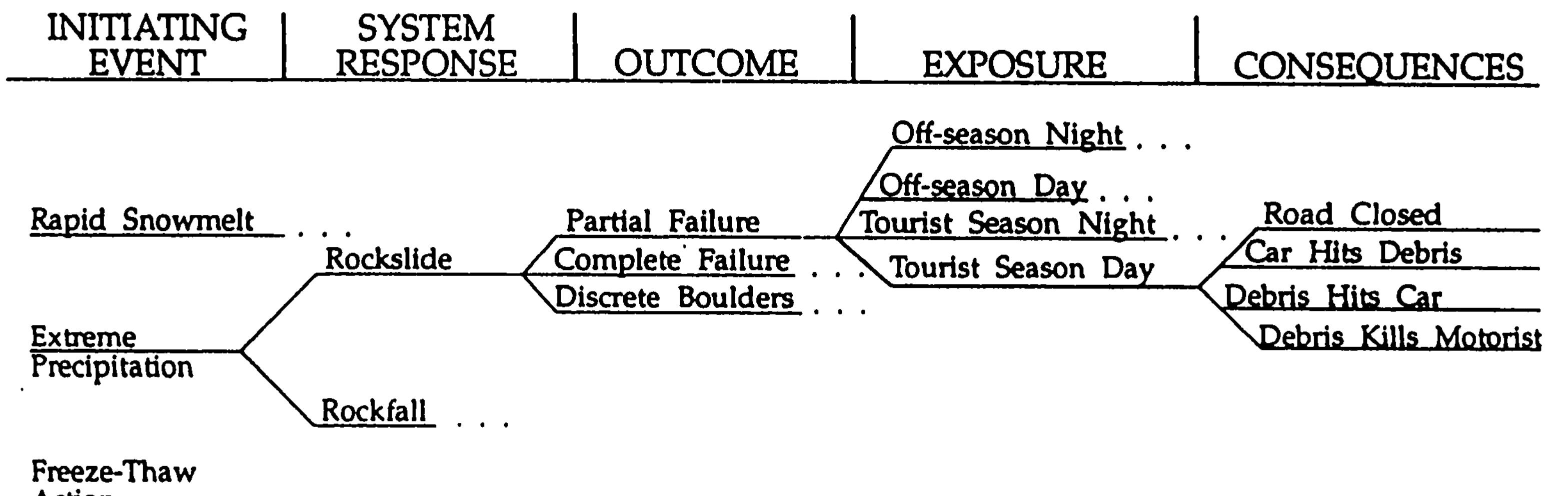
4.3.3 Rosenblueth's Method

Rosenblueth's method of point estimates is an approximate numerical integration approach. The expected value of any variable F is found by adding several terms. These terms will be four if the basic variables are only 2, e.g., c and ϕ. Eight terms are required if the basic variables are only 3, e.g., c, ϕ and pore water pressure r_u. Finally, sixteen terms are used if the basic variables are only four, e.g., c_1, ϕ_1 the shear strength parameter on the basal slip and c_2, ϕ_2 the shear strength parameters on the rear slip plane in a slope stability problem involving a wedge type failure mechanism.

This method is convenient to use and has proved to be successful. It does not require the calculation of the derivatives of F and can, therefore, be used in any geotechnical and especially slope stability problem regardless of how complex the expression for F is. It is noted that both Taylor series and Rosenblueth methods include the correlation between variables. In contrast, Monte Carlo simulation technique appears to have been used only for independent (uncorrelated variables)until recently (Chowdhury 1984).

4.4 EVENT TREE:

An "event tree" may represent the actual risk model. It can be used to show the many sequences of events that must be considered in order to evaluate the pathway probability of a certain outcome occurring. Figure 4-2 shows a hypothetical event tree branch, as prepared by Anderson et al.(1996), for evaluating the outcome probability for a landslide risk assessment. The model for this tree was based on a landslide



Note: ". . ." means the tree continues but is not shown due to lack of space.

Figure 4-2. Hypothetical event tree branch for evaluating outcome probability for landslide risk assessment. (After Anderson et al 1996) investigation made by R.T. Pack in 1994. Event tree was also used to show the various pathways by which the geotechnical failures can occur and to demonstrate how pathway probability can be calculated.

Developing the risk model is a critical part of the risk assessment procedure and involves an event tree similar to the branch shown in Figure 4-2. The development commences with identification of sequences of events that can initiate failure, and ends with the sequences of failure. Initiating events in the intermountain region of the US might be classified as rapid snowmelt, long term precipitation, a cloudburst or earthquake. Other type of events such as hurricane or typhoon, may need to be considered in the other parts of the world. Low-level, high probability events that normally would not lead to a landslide are included because a risk assessment considers a full range of loading conditions even if the system response is no failure.

The initiating event results in a number of system responses that might include debris slide, debris flow, slope detachment, ground cracking, or no failure. Each system response, including no failure, must be assigned a conditional probability of occurrence given that the initiating event occurs. The system response probabilities, including the no-failure response probability must sum to one for all of the branches from each initiating event. The probability of a given system response for a given event will require evaluation of a number of factors, such as the probability that the pore water conditions will be at some critical level at the time of initiating event.

A given system of response may then lead to a variety of outcomes. Two examples of a possible outcome are (1) debris slide mobilization into a debris flow or (2) a landslide of a given run out distance. The outcome category provides flexibility in the use of the risk assessment model and should be adapted to specific situations.

Initially, all possible system responses should be included in the event tree. After the initial tree has been developed, it may be desirable to combine some of the system responses. The decision to combine two or more of the system responses will depend on the independence of the events.

The individual pathway probability of realizing a certain outcome is the product of the event probability, the conditional probability of the system response given the event, and the conditional probability of the outcome given the system response. The pathway probability of the i^{th} pathway, P(Pi), is obtained as follows:

$$P(Pi) = P(E)\ P(F/E)\ P(O/F) \qquad (1)$$

in which P(E) equals the annual probability of occurrence of loading in a range of magnitudes, E; P(F/E) equals the conditional probability of the system response, F, in the i^{th} path given the loading occurs in the range E; and P(O/F) equals the conditional probability of the outcome, O, given that the system response, F, occurs as a result of the loading in the range E.

The magnitudes of the resulting life loss, property damage or environmental damage will depend on various exposure factors. These can be defined by determining the path of the resulting outcome such as a soil slip, debris flow or mud flood, and then considering the travel time, time of day, season of the year and the effectiveness of any warning system and accompanying evacuation plan.

Consequences are classified as life loss and economic loss, which include property damage, cost of dislocations, and lost benefits. The partial risk costs, Ci, for the i^{th} pathway is obtained by taking the following:

$$Ci = P(Pi)\ P(L/O)\ P(C/L)\ L_E \qquad (2)$$

where: P(L/O) is the conditional probability of the conditional probability of property damage given this exposure, and L_E is the resulting economic consequences.

Risk cost is an annual average (or expected) value of damage associated with event sequences described in the risk model. The total risk cost C is obtained by summing the partial risk costs over all N mutually exclusive pathways in the event tree.

$$(C = \sum_{i=1}^{n} Ci)$$

The expected magnitude of life loss, LL, from a population at risk of magnitude, PAR, due to a failure described by the i^{th} pathway is given by:

$$LL = P(Pi)\ P(L/O)\ P(C/L)\ PAR \qquad (3)$$

In this case, P(C/L) is the conditional probability of loss of life given the exposure to the outcome. In addition to the loss of life issues, environmental and social consequences can also be considered.

4.5 EXAMPLE FOR APPLICATION OF PROBABILISTIC ANALYSIS. (Probabilistic Analysis of Swiss Landslides).

The following study was presented by Oboni et al. 1984. The basic objective of the project entitled DUTI (Detection and Use of Landslide Prone Areas) include the development of methodologies for risk mapping. This study describes at first the basic principles, characteristics and requirements of anew slope stability approach and then comments the results obtained in three typical site studies selected in Switzerland and thoroughly monitored within the DUTI project.

4.5.1 Probabilistic Stability Analysis

Natural and man-made slopes have been analyzed by the DUTI research workers. At the basis of the used approach, five statements can be formulated, drawn from the experience in observing actual slide phenomena in the Swiss Alps:

1) The degree of mobilization of the shear strength can hardly be considered as a constant along a given slip surface; so all classical methods should be avoided or at least their

limitations should be taken into account in the evaluation of their results.

2) Circular or nearly circular slip surfaces are very uncommon in natural slopes.

3) Macroscopical evidence of a slide can only be observed if the toe of the slope fails. Tension cracks and deformations occur over years on a slope without reaching a catastrophic stage, if the toe is able to resist the forces transmitted by the upper part of the sliding mass. If the toe happens to fail, then consecutive displacements of the mass will occur.

4) Once the toe has moved, the failure can progress toward the top until it reaches a more stable zone which acts as a retaining structure in regard to the upper part.

5) Describing the actual conditions of natural slopes, one has to select a slope stability index taking into account the dispersion of the mechanical parameters of the soil layers and the water table fluctuations. A suitable stability index is therefore the so-called probability of failure.

4.5.2 Main Features of the Method

The profile of the slope is divided in n vertical slices. Each one slides on a segment of the slip surface characterized by a cohesion parameter c_i and a friction parameter ϕ_i that are random variables; the Mohr-Coulomb failure criterion is assumed for all materials.

The first and second moments - mean and variance - of the capacity C_n (shear resistance) and of the demand D_n (shear driving force) are computed for the n^{th} element of the slope, i.e. the top element, using the Rosenblueth point estimate method. The analytical formulation of local capacity and demand is given at the end of this example. This element will transfer to the $(n-1)^{th}$ a compressive random force, the value of which is $P_n = D_n - C_n$. The first and second moment of the capacity and the demand the antecedent element, $C_{(n-1)}$, and $D_{(n-1)}$, can now be computed introducing the transmitted force P_n in the equilibrium equations. This repetitive procedure is continued until the toe slice is reached.

For each slice, a local probability of failure is then determined: it expresses the probability that a given slice will fail if the one below has already ceded. Therefore, having reached a given slice, failure may or may not progress to the upper element, according to its own conditions of equilibrium.

The results are expressed mainly by two diagrams drawn on the abscissa axis of the profile; the first one gives the values of the local probabilities of failure or transition probabilities, p_f and the second one the mean values of the transmitted forces, $\overline{P}_I$.

At this stage, an important aspect of the method has to be emphasized. Although the calculations are carried out from the top to the toe, the resulting probabilities to be understood as the transition probabilities of the random process reflecting the regressive nature the failure phenomenon.

4.5.3 Geotechnical Data for the Stability Analysis

Generally, only poor or limited information about the subsoil conditions can be obtained in preliminary studies of large landslides. Indeed, the large depth of the sliding masses, complex geological structures and high cost of drilling in difficult topographical conditions oblige to limit investigation programs and the number of boreholes as well as the laboratory tests.

The budget available for a study is often related to the economic importance of a particular landslide, which depends greatly on the use of the site. In mountain regions, where the risks do not seem catastrophic, the social cost of this phenomena does generally not justify an exhaustive study in order to design a suitable stabilization work. That is the reason why during long periods of time between sudden accelerations, no preventive actions is taken. The slowly damaged structures and roads are simply repaired without any quantification of the probabilities of failure and of the involved risks.

For the site studied, it was possible to drill in general three to ten boreholes per landslide, reaching total length of 100 to 300 m. However the number samples taken to the laboratory for standard geotechnical testing was often small, either because of the nature of the materials or due to the limited availability of the testing facilities. The number of shear tests rarely exceed ten, for example. In these conditions, statistical analysis of data was not possible. Yet, the basic characteristics of the distributions could be adapted or corrected by the experience gathered on similar soils.

The average values of the in situ material characteristics can be supposed to be equivalent to the average values obtained from the samples. As far as variances are concerned, the test values must sometimes be weighted so that the coefficient of variation of the cohesion and of the tangent of the friction angle remains in the typical range for the concerned materials. Then, the minimum and maximum values necessary to define the Beta distribution of the parameters can be chosen based on probabilistic or physical meaning criteria.

Finally, by means of boreholes equipped with piezometers and inclinometers, water table fluctuations measured by discrete values or continuous recording with a gauging stationary observed during one to two years. The interpretation of these measurements and of some historical data leads to the definition of a possible dispersion of the water table level.

4.5.4 Results of Site Studies

Three very different landslide phenomena have been selected to present the results given by the probabilistic stability analysis. La Frasse landslide covers an area of more than 1 km^2 in the Swiss Prealps. The second site is La Chenuala landslide, situated near Lusanne, is smaller, but has experienced a very remarkable acceleration in 1983. Finally Le Day landslide in the Jura slopes, a low mountain formation in western Switzerland, is crossed by a major road and a railway track.

4.5.4.1 La Frasse Slide

La Frasse landslide is a well known example in Switzerland. Two main roads of touristic interest cross the area are regularly damaged by the movements. There is a small human settlement and several isolated houses on the site. Some chalets, built in XVII and XVIII centuries, present marked deformations due to the constant sliding activity which shows, moreover, periodical reactivations.

The right bank of the Grange Eau river, which flows at the toe of the landslide, is cut into a vast syncline made of rigid calcareous rocks. The axis of this syncline is parallel to the valley and its heart is filled with Flysh. The stability of this slope is generally excellent because of the overturned side of the syncline acts as retaining wall for the Flysh. However, this stabilizing mass was destroyed on a length of about 800 m, probably by the glacial activity, so that the Flysh began to slide. Some 2000 m^3 of slidden materials are therefore evacuated each year by the floods of the Grande Eau river below. In general, the body of the slide has a fairly smooth surface, except in some areas which abound in tensile cracks.

According to the meteorological conditions, the slide can be subjected to catastrophic reactivations which destroy roads and severely damage houses and electrical power supplies. During crisis periods, the analysis of the movements in different parts demonstrates that velocity increases by steps from the top to the toe of the slope.

From the geotechnical point of view, the materials of the sliding mass are defined as very clayey sands or gravels in alternating structures derived from the remolding of Flysh formations. These materials are generally not sensitive to variation of the water content. The geotechnical parameters taken into account for the stability analysis are shown in table 4-2.

	Cohesion [$KN.m^{-2}$]	Friction [°]	Density [$KN.m^{-3}$]
Mean value	22	28	22
Standard deviation	9.5	7	--

Table 4-2. Geotechnical parameters of La Frasse sliding mass (After Oboni et al. 1984)

Figure 4-3 gives the results of the probabilistic stability analysis, which are compared with the safety factors of the classical Morgenstern and Price method. The interpretation of the results obtained by Oboni is mainly the following:

a) Considering simultaneously the transition probabilities and the mean transmitted forces diagrams, it can be observed that at the abscissa x = 600 m, P is nearly naught and P_f grows drastically as the abscissa decreases. This means that the lower part of the slide behaves almost independently with respect to the above laying mass. Tensile cracks should appear

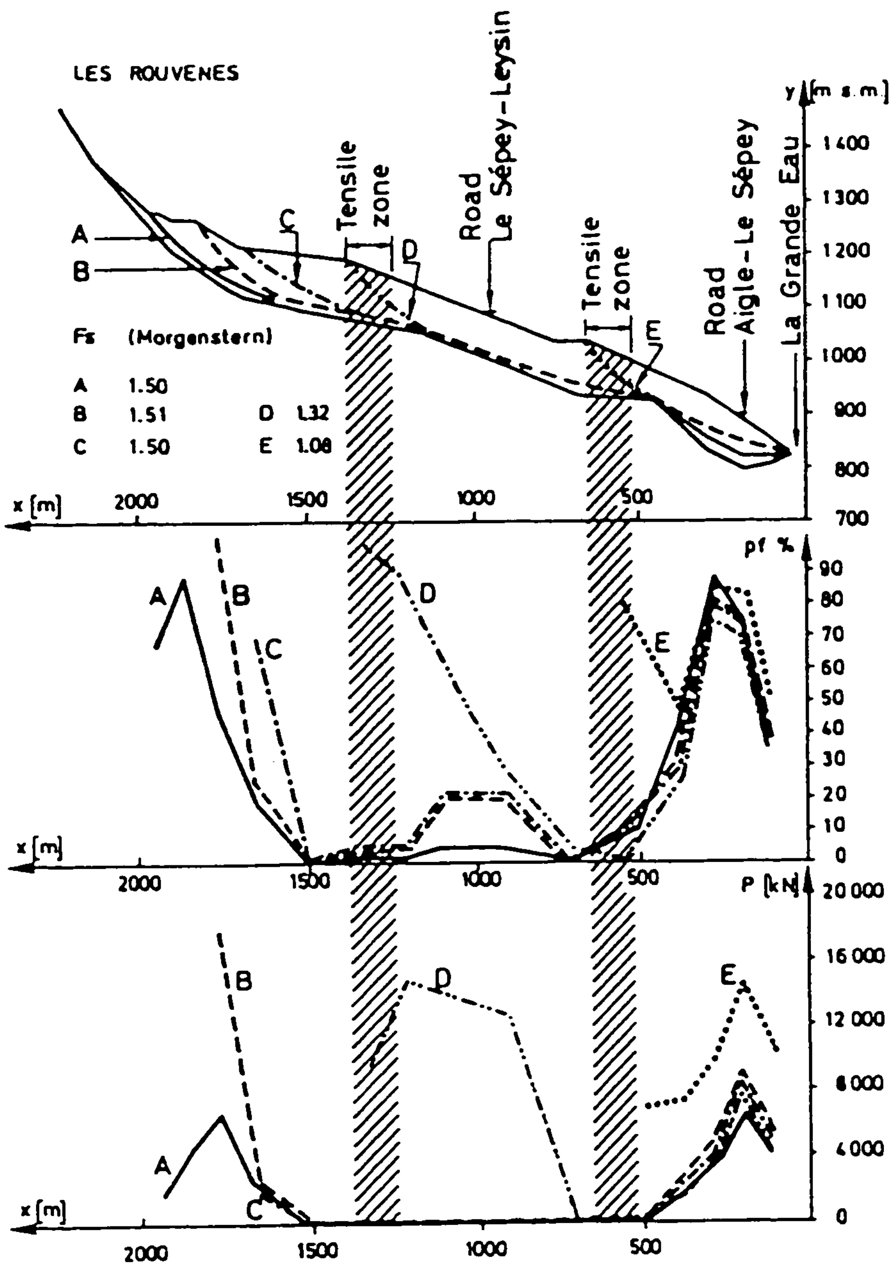


Fig 4-3 Probabilistic versus deterministic results for La Frasse landslide.(after Oboni et al. 1984)

immediately uphill of the x = 600 m limit. The same phenomena, though of lower intensity, appears at x = 1300 m.

b) Between x = 700 m and x = 1500 m, the mean transmitted forces are of relatively low intensity, whereas the transition probabilities reach a value of about 0.2. This area is therefore unstable by itself and retained by the rigid step at x = 700 m. At this abscissa, the transition probabilities are low (0.05), but the retaining capability of this section is not large enough to avoid the sliding.

c) The section between x = 0 and x = 500 m presents the typical features of a very sensitive slope in limit equilibrium conditions. The probability of initiating the failure is 0.4. A very small flood of the Grande Eau river can under these conditions generate a sudden reactivation of the slide, reaching by progression the abscissa x = 500 m and producing tension cracks immediately uphill. The results obtained by this analysis appear to be in very good agreement with the actual geological and geodetical observations.

4.5.4.2 La Chenaula Slide

The Chenaula landslide is located 5 km East from the city of Lusanne. The slope is entirely covered by a high forest and totally uninhabited but crossed by a main state road. Being close to Lusanne, it was easy to observe it closely during almost 30 months and monitor its slow movements. At the end of this period, the slope failed. The displacement reached 15 m in three days.

The first slide on La Chenaula is certainly several thousand years old and its extension is bigger than the today's active zone. The moving mass is made of clayey glacial deposits, strongly disturbed, sliding on the bedrock. Its surroundings present a number on similar phenomena, induced by the generally poor quality of the molassic bedrock (sandstones and marls in layered structures, dipping parallel to the slope).

During the spring of 1936, a big slide occurred on the opposite bank of the Chandelar stream which flows at the toe of La Chenaula and pushed the river against the considered slope. Moreover, the stream has grown its erosion capability during these last years. That is mostly by the construction of new housing uphill that spill their clear water into the river. The amount of materials eroded by the river in the slide stretch, under average flow conditions, has been evaluated to 70 m^3 per day.

As far as site investigations are concerned, piezometric - discrete and contiguous - measurements show very unfavorable hydrological conditions. The water level is immediately under the slope surface and even artesian at some places. The number of inclinometers implemented in the slide allowed the determination of the slip surface with a very good likelihood and also the plotting of cumulative displacement graphs at different depths and locations.

The stability analysis was carried out in three steps:

a) unfailed slope, under c_u, Φ_u conditions;

b) unfailed slope under residual conditions;

c) failed slope under residual conditions.

Figure 4-4 shows the transition diagrams for these three models. Attention must be paid to the drastic increase of the values of the transition probabilities between case a and b at the toe of the slide. After sliding, the slope is of course safer than before, but a new peak of transition probabilities appears near the abscissa x = 100 m. The transition diagrams yielded by the method allow the understanding of failure mechanism and can be used to define the best location for remedial works.

4.5.4.3 Le Day Landslide

This landslide develops in a very dense forested area and is crossed by the international Milan - Paris railroad and by a main state road. After years of periodical superficial maintenance and execution of shallow drainage channels, the national railroad administration decided in 1981 to undertake a general study of the phenomenon and of the possible remedial works in order to stabilize the site, for which a private consultant and the DUTI project research workers were chosen.

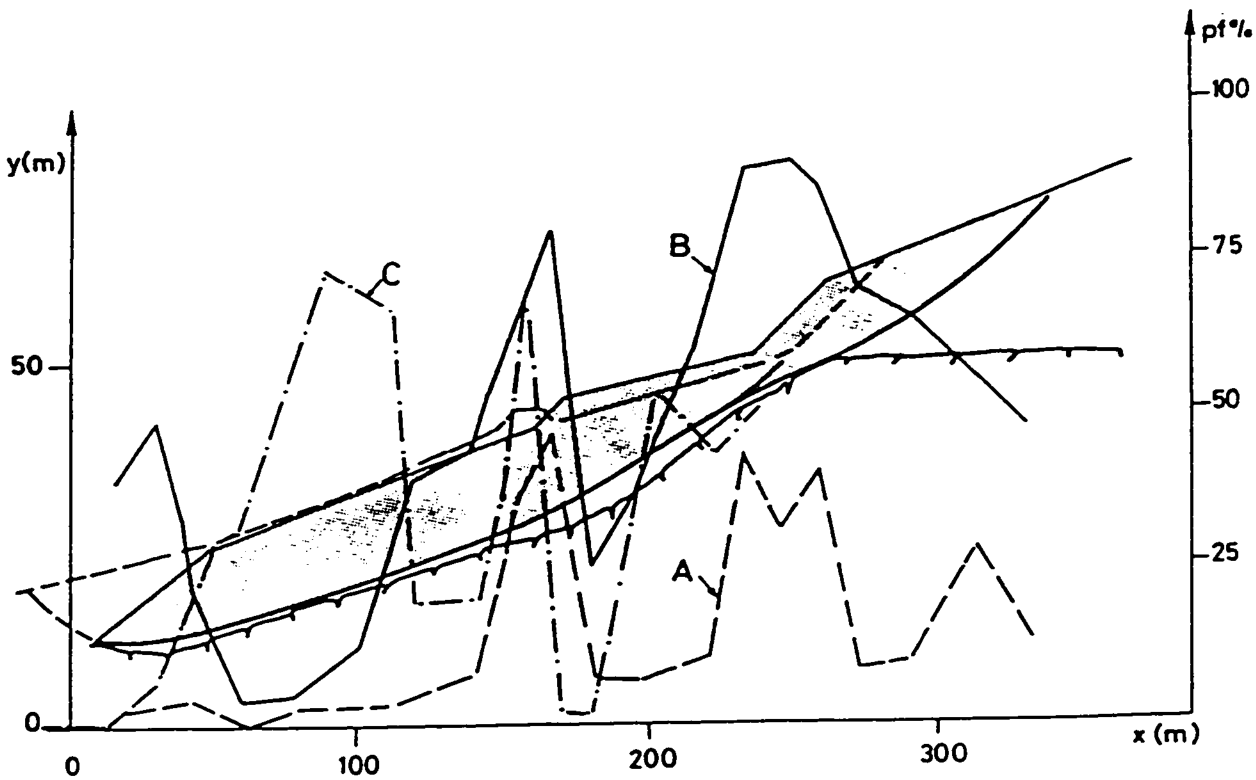


Fig. 4-4

Probabilistic analysis of La Chenaula landslide, transition diagrams. (After Oboni et al. 1984)

a) unfilled slope under c_u, Φ_u conditions

b) unfilled slope under residual conditions

c) failed slope under residual conditions.

This slide consists in a very dense rhodanian moraine and glaciofluvial sediments covering glacial lake deposits and moraine debris. These formations fill the broad syncline made of Jurassic and cretaceous rocks (mainly limestone and marls). The axis of this syncline is approximately parallel to the bed of the Orbe river that flows at the toe of the slope. The main cause of the slide initiations probably the fluvial erosion immediately after the last glacial era.

The top of the slide is characterized by a long tension crack (more than 1 km) bracing the whole landslide. At the railroad level, three different areas with distinct velocities can be defined. Each one presents secondary tension cracks indicating the various failure surfaces exist in the slope, one or two of which have been identified by inclinometric measurements.

As shown in Figure 4-5, the probabilistic model for the stability analysis indicates that a tensile crack at the abscissa of the railroad should appear in natural conditions, what is actually happening now. The DUTI project intended also to qualify the possible performance of remedial works, using prestressed anchors of a length of about 80 m which develop a force of 880 kN/m, as computed by the private consultant. The attention is focused here on the interpretation of the transition programs established both for the natural and anchored slope.

Figure 4-5 revealed that the transition probabilities in the sector of the railway are about 0.1 after anchoring, whereas they reached about 0.3 before. Although it is not known whether the railway company can afford this 0.1 probability, it must be pointed out that the designed remedial works improve the stability of the railroad section by a factor of 3.

Furthermore, the mean transmitted forces diagrams is modified by the introduction of the anchor, so that even the stability of the toe of the slide increases.

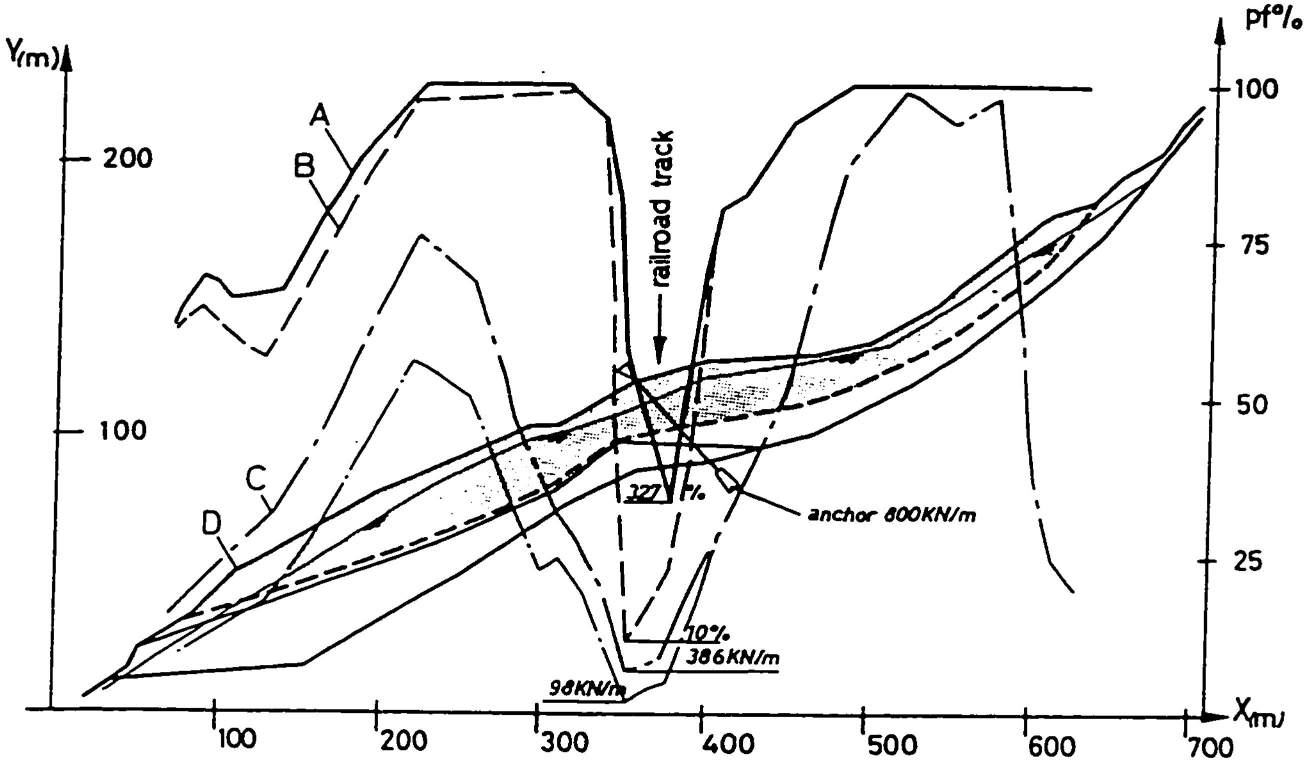


Fig. 4-5 Le Day landslide

($\bar{c}$ = 23.8 $kN.m^{-2}$, S_c = 8.6 $kN.m^{-2}$, $\bar{\Phi}$ = 160°, S_Φ = 40°)

A = transition diagram without anchors

B = transition diagram with anchors

C = mean transmitted forces diagram without anchors

D = mean transmitted forces diagram with anchors.

<u>4.5.5 Evaluation of Capacity and Demand for the i^{th} Element of the Slope</u>

In Figure 4-6, The following terms are identified:

W_i = weight of the i^{th} slice $[kN.m^{-1}]$

$W_i' = W_i - \gamma_w\ h_i\ b_i$ $[kN.m^{-1}]$

$S_i = \sin(\eta_i)\ \gamma_w\ h_i\ b_i$ $[kN.m^{-1}]$

So that:

$S_{//i} = S_i \cos(\alpha_i - \eta_i)$

$S\perp_i = S_i \sin(\alpha_i - \eta_i)$

$P_{//i} = P_{i+1} \cos(\alpha_i - \eta_{i+1})$

$P\perp_{i+1} = P_{i+1} \sin(\alpha_i - \eta_{i+1})$

C_i and D_i are computed as follows:

$C_i = (W_i' \cos\alpha_i - S\perp_i - P\perp_{i+1})\ .\ tg\ \phi_i' + c_i'\ b_i / \cos\alpha_i$

$D_i = Wi' \sin\alpha_i + S_{//i} + P_{//i+1}$

The i^{th} element transfer a random force P_i to the i-1^{th} element:

$P_i^* = D_i = C_i$

with :

$P_i\ (C_i,\ D_i) = P_i^*$ if $(D_i - C_i) > 0$

$P_i\ (C_i,\ D_i) = 0$ if $(D_i - C_i) < 0$

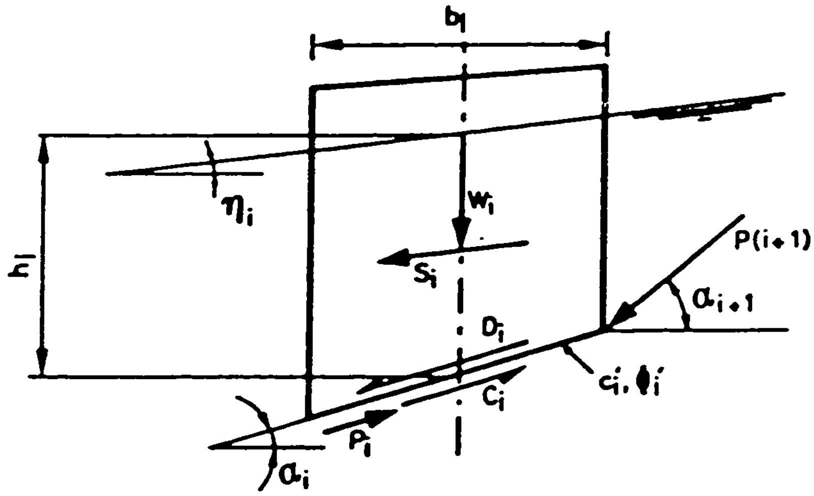


Fig. 4-6 Forces acting on the slice I

(After Oboni et al. 1984)

Chapter: 5

RISK ASSESSMENT

In this chapter, we introduce a model for ranking landslide intensity. This model takes into account most factors that cause landslides.

5.1 HISTORICAL RECURRENCE OF LANDSLIDES

In addition to the factors mentioned in the previous chapters, another important factor is also considered here. That is the previous occurrence record. History of landslides shows a clear relation between previous occurrence of landslide and potentiality for new ones. Statistics of past landslides can provide a guide for the possibility of future landslides.

This is to mean that the higher the frequency of slides, the higher the probability is for future landslides. This fact was emphasized by Peck (1967) when he showed that if there is no evidence of old landslide in an area, then it is fairly unlikely that moderate construction activity will start a new one. On the other hand, if old landslides abound, then it is likely that even minor construction will activate a new one.

5.2 RISK INTENSITY

As discussed in chapter one, Factor of Safety on slopes was the major guide for slope stability evaluation. Recently, landslide studies have introduced analytical and probabilistic methods for landslide studies. There are two main reasons to use risk assessment rather than the factor of safety.

First, the factor of safety gives information on a certain area while risk assessment can provide a more global view of the problem. Secondly, safety factor takes into consideration the geotechnical conditions only such as slope geometry, soil properties and pore water pressure. Whereas, in this study, risk assessment considers also environmental and human factors that may enhance slope instability as well as the statistics of past landslides.

In the previous chapters, it was shown that there are various factors that can affect landslide probability. These are the safety factor, the environmental conditions, human activities, slope protection means and landslide history. It is clear that these factors are independent to each other. The absence of any one will not prevent landslides, but presence of any of one or more will be enough to have a possible landslide.

Thus, the relation between landslide possibility and its affecting factors is similar to the union relation ($\cup$) in probability. This means that the probability of landslide is the union of the probability of the factors which affects landslide occurrence. Accordingly, the landslide factor may be taken as the sum of the factors related to landslide causes.

Based on the above, the following model is proposed for risk assessment. Each factor is given a grade according to its effect of slope instability. All of them are aggregated in a way to give maximum risk at worst condition of each factor and minimum risk at the best condition of each factor. The model was applied to a case study in Repentigny and proved to be realistic. Details of this study is given in chapter five.

Risk factor can be identified using the following formula:

$$X = A + B + C - D + E \tag{5-1}$$

Where:

X= Risk intensity (ranked from one to twelve),

A= Grade related to the slope factor of safety (F.S),

B= Factor related to the natural environment,

C= Factor related to human activities,

D= Factor related to site modifiers intended to control landslides,

E= factor related to previous occurrence of landslides.

Fig. 5-1 shows a proposed procedures for risk assessment.

In the following, identification of the values of the factors B, C, D and E is explained. It is important to mention that all used figures are arbitrary parameters for the purpose of relative ranking. These parameters and limits may be adjusted to regional conditions. In addition, a weight factor may be introduced to emphasize the importance of certain factors if so justified.

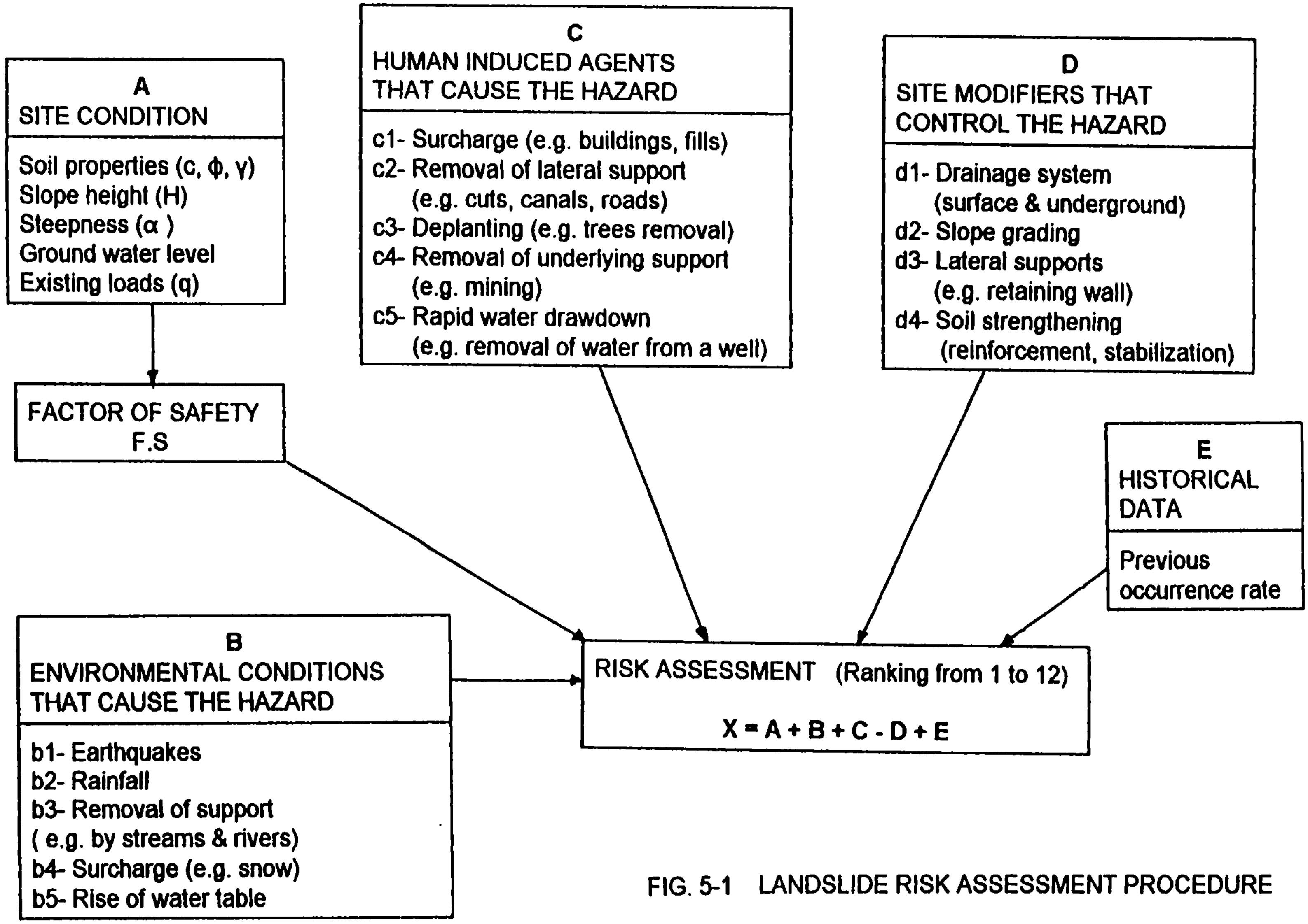


FIG. 5-1 LANDSLIDE RISK ASSESSMENT PROCEDURE

Another fact, which was taken into consideration, is the relativity in hazard perception. As explained in Chapter one, what is considered as hazard for some people may be not for others. Therefore, factors may be graded according to its position in the range. For instance, either to be within the mid-third of range, the first third, or the last third of range. This approach was used by (Varnes, 1984) in his guide to UNESCO zoning for landslides.

A: GRADE RELATED TO THE GEOTECHNICAL FACTOR OF SAFETY (F.S)

Value of (A) shall be classified according to F.S as shown on table (5-1).

Factor of Safety	A
< 1,25	4
1.25 - 1.5	3
1.51 - 2.0	2
>2.0	1

Table 5-1: Grading of Factor of Safety value

B: FACTOR RELATED TO ENVIRONMENTAL CONDITIONS:

B shall be taken as the highest of b_1, b_2, b_3, ... as follows:

b_1: Earthquakes Factor

b_1 shall be classified according the intensity of earthquakes as shown in table (5-2).

Probability of occurrence in high intensity	b_1	possible scale
within lower third of range	3	>6 Richter
within mid-third of range	2	4-6 Richter
within upper third of range	1	< 4 Richter

Table 5-2: Grading of earthquakes condition

b_2: Rainfall rate factor

b_2 shall be identified according to the intensity of rainfall rate as shown in table (5-3).

Rainfall rate (average)	b_2
within lower third of range	3
within mid-third of range	2
within upper third of range	1

Table 5-3 Rating of rainfall rate

<u>b_3: Removal of support</u>

For instance, erosion by streams and rivers.

Removal of support	b_3
within lower third of range	3
within mid-third of range	2
within upper third of range	1

Table 5-4 Removal of support factor

<u>b_4: Surcharge load</u> :   For instance, snow and load.

Surcharge load (average)	b_4
within lower third of range	3
within mid-third of range	2
within upper third of range	1

Table 5-5 Surcharge load factor

<u>b_5: Rise of water table</u>

Water table rise	b_5
within lower third of range	3
within mid-third of range	2
within upper third of range	1

Table 5-6 Rise of water table factor

C: FACTOR RELATED TO HUMAN INDUCED AGENTS

C= The highest of c_1, c_2, c_3, . . . which are identified as follows:

c_1: Surcharge: (for instance, construction or fills)

c1 shall be identified according to the expected surcharge load as shown in table (5-7).

Expected surcharge load	c_1
high	3
moderate	2
low	1

Table 5-7 Surcharge induced by human

c_2: Removal of lateral support: (Cuts, canals, roads)

c_2 shall be identified according to the expected removal of lateral support as shown in table (5-8).

Removal of lateral support	c_2
very likely	3
likely	2
unlikely	1

Table 5-8 Removal of lateral support by human

c_3: Deplanting : (removal of trees and vegetation)

Deplanting	c_3
uncontrolled	3
moderately controlled	2
well controlled	1

Table 5-9 Deplanting (human induced)

c_4: Removal of underlying support: (for instance, mining)

Removal of underlying support	c_4
uncontrolled	3
moderately controlled	2
well controlled	1

Table 5-10 removal of underlying support

c_5: Ground water table movement, rapid drawdown:

This may be induced by removal of water from canal, reservoir or well. c_5 shall be graded as shown in table 5-11.

Rapid drawdown	c_5
uncontrolled	3
moderately controlled	2
well controlled	1

Table 5-11 Rapid drawdown factors

D: MITIGATION METHODS THAT AFFECT OCCURRENCE OF LANDSLIDES

D= the highest of d_1, d_2, d_3, . . . As follows:

d_1: Drainage system (surface and underground)

A well designed drainage system can control the moisture content of soil. As a result, the effect of change to soil shear strength can be minimized. Drainage system factors shall be identified according to the following table.

Drainage	d_1
good	3
moderate	2
poor	1

Table 5-12 Drainage system factors

d_2: Slope grading:

By grading of slopes, shear stress can be reduced and stability of slopes will be improved.

Grading of slopes	d_2
well graded	3
moderately graded	2
steeply graded	1

Table 5-13 Slope grading factors

d_3: Lateral support: (Examples: By use of retaining structures, shear stress can be reduced.)

Lateral support	d_3
well integrated support	3
moderately integrated support	2
no support at all	1

Table 5-14 Lateral support factors

d_4: Soil strengthening (For instance, reinforcement, pining, stabilization, piling, or planting.)

Soil reinforcement	d_4
well reinforced	3
moderately reinforced	2
not reinforced	1

Table 5-15 Soil reinforcement factors

E: HISTORICAL DATA: (Previous frequency of slides enhances the possibility of reoccurrence.)

Previous occurrence	E
frequent	3
moderate	2
rarely occurred	1

Table 5-16 Historical data factors

5.3 COMMENTS

By using formula (5-1) for risk assessment, we can get different grades from one to twelve. For example in a situation where the factor of safety low, with a severe natural conditions, uncontrolled human activities, no protection works, and a record for past frequent landslides, we can get the following factors:

$A=4, \quad B=3, \quad C=3, \quad D=1, \quad E=3$

then from equation 5-1:

$$X = A + B + C - D + E = 4 + 3 + 3 - 1 + 3 = 12$$

On the other hand, where we have a high factor of safety, light environmental conditions, controlled human activities, suitable slope protection, and record of minimum past occurrence of landslides, we will have the following factors:

$A=1, \quad B=1, \quad C=1, \quad D=3, \quad E=1$

Then from equation 5-1:

$$X = A + B + C - D + E = 1 + 1 + 1 - 3 + 1 = 1$$

Any other condition will be between one and twelve. In general, we would expect higher possibility of landslide with the higher the value of X.

Chapter: 6

CASE STUDY

(City of Repentigny)

Repentigny is a town located to the north east of Montreal, Quebec. Fig. 5-1 shows the geographical location of Repentigny. The town has been subjected to a number of landslides along L'Assomption river over the years. The latest was in April 1996. Most of the slides took place during the spring season.

In a meeting with Mr. Denis Tremblay, the City Engineer, he reported that landslides have been a continuous problem in Repentigny all through his 23 years of services. Chagnon (1982) and Demers (1996) have concluded that the slides were mainly due to erosion. The fact that L'Assomption river has a significant change of direction, with severe curvature at this area, can be a reason of erosion and removal of underlying support of the river bank. .

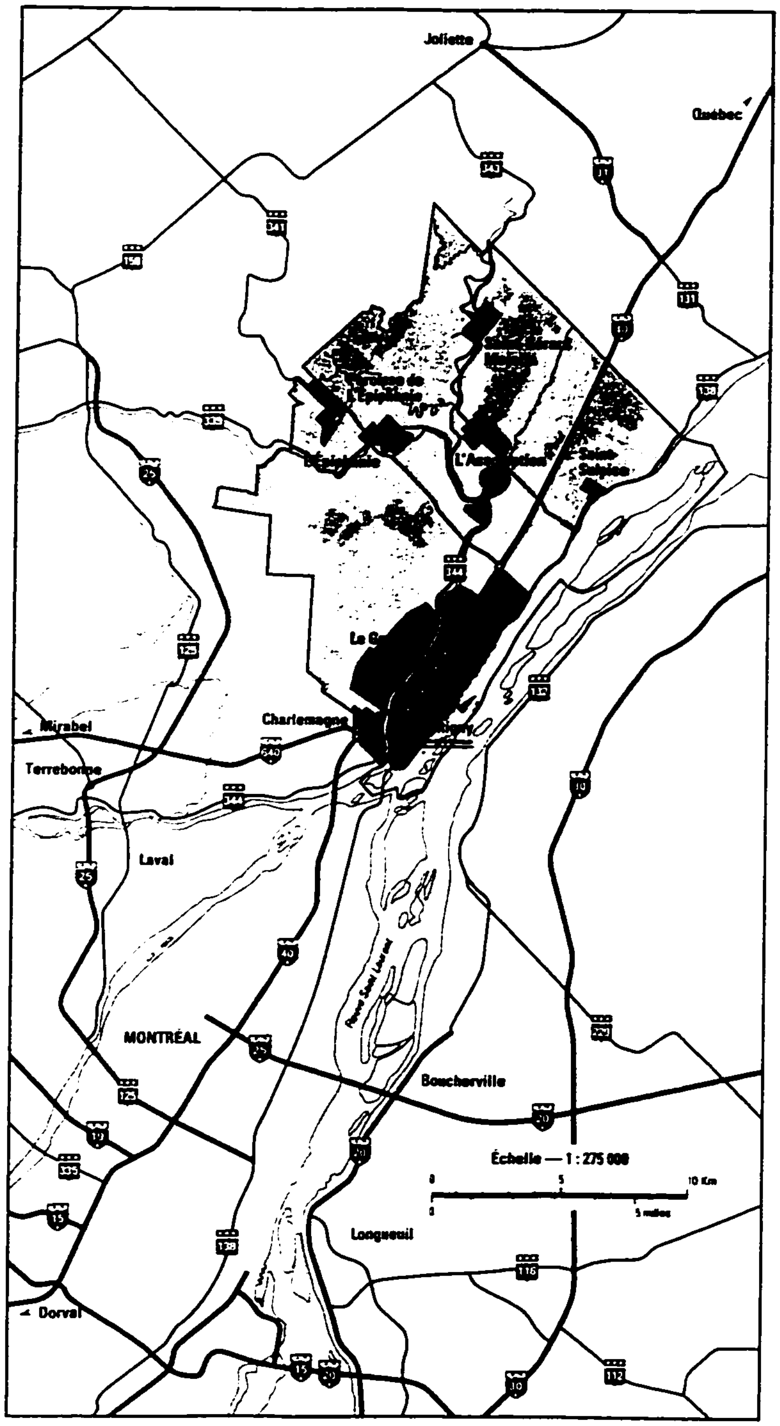


Fig. 6-1 Location of Repentigny.

6.1 RISK ASSESSMENT

A- FACTOR OF SAFETY

In a geotechnical study made by Transport Quebec (Demers, 1996), the factor of safety was determined to be 1.04, 1.01, and 0.99. Using the average of this values for the general condition, we can get:

$$F.S = (1.04 + 1.01 + 0.99) / 3 = 1.01$$

From table 5-1, A can be determined as: $A = 4$ (6-1)

B- ENVIRONMENTAL CONDITIONS:

B-1: Earthquakes factor:

Information from the city personnel indicated that Repentigny experiences low probability of earthquakes. This can be classified in the last rank of table 5-2 to give:

$$b_1 = 1$$

B-2 Rainfall rate

In a telephone conversation with Mr. Roger Gagné from Environnement Quebéc on 28/5/96, he stated that there is no observation station in Repentigny. The closest is the observation station of Rivière des Prairies which can be used as a reference for this study. The average annual rainfall, from statistics of Environnement Québec, were reported as given in table (6-1).

Station	average rain fall rate (mm/year)
Riviere des Prairies	800.7
Mascouche	814.8
St-Hubert	779
Montreal	739

Table 6-1 Annual rainfall rate around Repentigny

Looking to the average rainfall during the month of April, in which most of Repentigny landslides took place, the statistics show that the average rain fall rate is as shown in table 6-2.

Station	average rain fall rate (mm/April)
Riviere des Prairies	72.8
Mascouche	69.7
St-Hubert	74.8
St-Amable	62.7
Dorval	72.8

Table 6-2 Average rainfall rate (during month of April) around Repentigny

From the above information and table 5-3, it is concluded that:

$$b_2 = 2$$

B-3 Removal of support

Erosion from L'Assomption river is a clear problem. Past studies on the Repentigny referred to erosion as the main trigger of slope failures along the river bank (Chagnon, 1971 & Demers, 1996). Erosion is a cause for removal of lateral support. Therefore, and using table 5-4, it is concluded that: $b_3 = 3$

Fig. 6-2 Landslide is a continuous problem in Repentigny

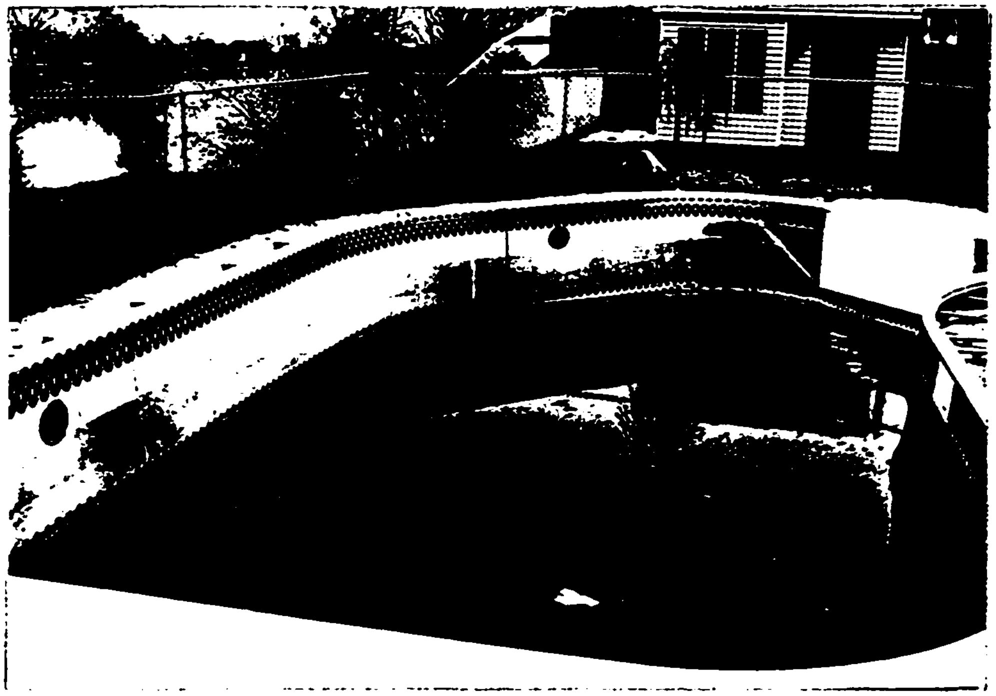


Fig. 6-3 Effect of latest landslide on residential buildings

B-4 Surcharge load

There in no exceptional surcharge can be traced in the area of the problem. The snow load can be easily compared to any other city in the area. Accordingly from table 5-5:

$$b_4 = 2$$

B-5 Rise of water table

Fluctuation of underground water level may be expected due to change of water level in the river. However, there is no indication that rise of water table is exceptional to similar locations. Therefore, using table 5-6, b5 can be taken as:

$$b_5 = 2$$

Taking the highest of b_1, b_2, b_3, b_4 & b_5, then:

$$B = 3$$

C- HUMAN INDUCED AGENTS

C-1 Surcharge

The area subjected to landslides is mostly residential buildings. This can be considered as moderate load and factor c_1 can be taken as : $c_1 = 2$

C-2 Removal of lateral support

Human removal of lateral support is not expected in this condition. That is because the slopes are generally to the river side.

$$c_2 = 1$$

C-3 Deplanting

Deplanting in the region can be considered as moderately controlled.

$$c_3 = 2$$

C-4 Removal of underlying support

As mentioned in c-2 above, no human activities are expected to affect the underlying support.

$$c_4 = 1$$

C-5 Ground water movement

Since there is no indication that this point has been controlled, c5 factor can be taken from table 5-11 as:

$$c_5 = 2$$

From c_1, c_2, c_3, c_4 & c_5, C=2

D- SITE MODIFIER THAT AFFECT OCCURRENCE OF LANDSLIDES:

Since there is no action has been taken yet, D factor will be taken from tables 5-12 to 5-15 as:

$$D = 1$$

E- HISTORICAL DATA

Landslides are repeatedly occurred in Repentigny.

Then; E = 3

Risk assessment: using equation 5-1:

$$X = A + B + C - D + E = 4 + 3 + 2 - 1 + 3 = 11$$

6.2 RISK EVALUATION

Risk evaluation, as used here, is to present the risk of hazard in a dollar value. Although this can not be done accurately, particularly when human life is involved, it is helpful in most cases. That monetary evaluation will give a better and easy understanding of the problem in a common used units. Two methods are proposed here to transfer the risk assessment grade, as devised in the previous chapter, to a dollar value.

The first method was presented by Petak, et al, (1982). It is more accurate but need a previous study of an algorithm factor each risk grade in different places. That is to devise a loss per person value to be expected for the various grades of risk. By knowing the population exposed to the hazard, a total loss value can be calculated. This approach is illustrated in Fig. 6-4.

The second method is simpler and based on analogy to probability studies. That is to transfer the risk assessment intensity to a probability factor. This can be done by dividing the actual risk grade by the highest grade (12). Then, by knowing the value of elements at risk, risk value can be obtained.

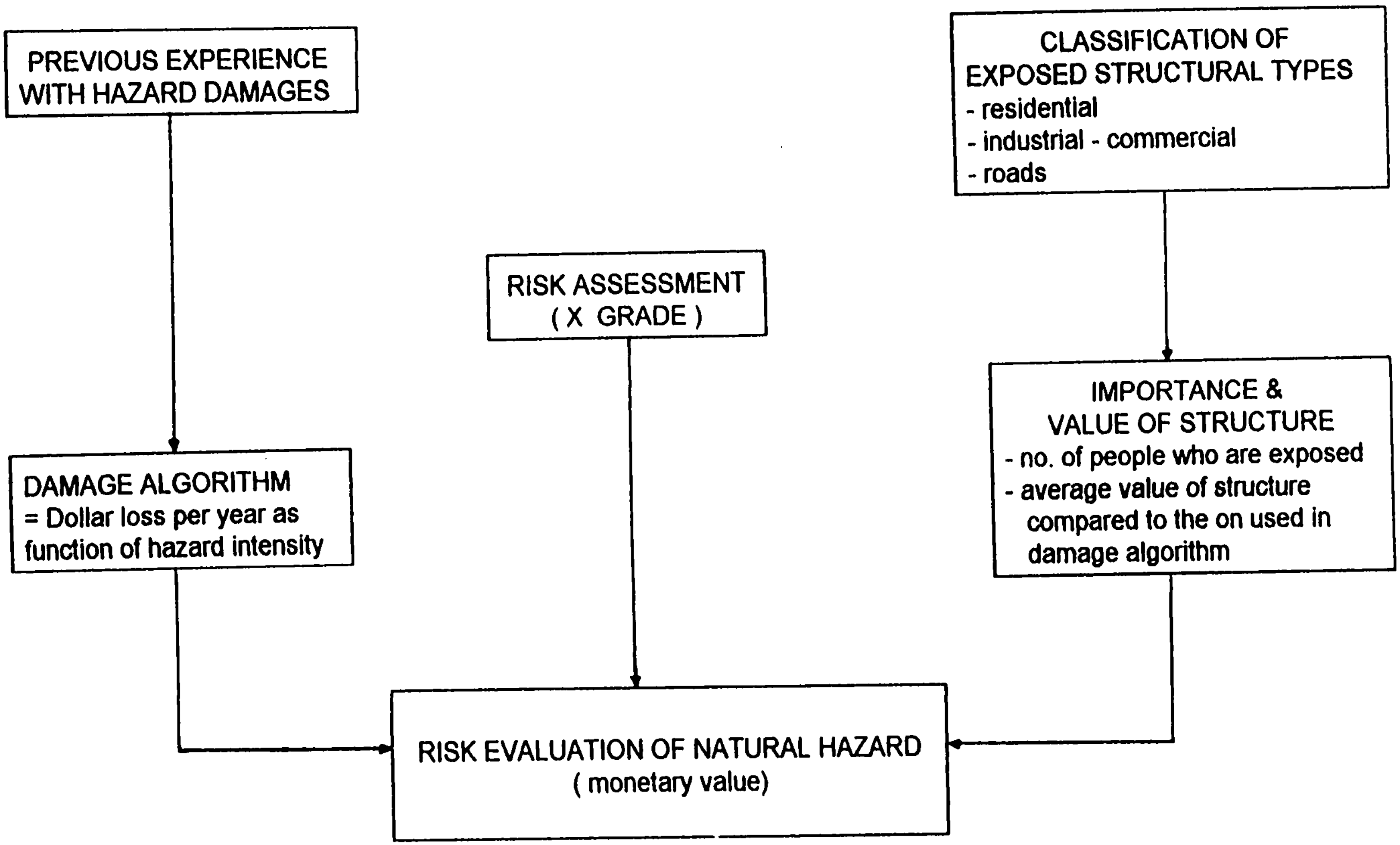


FIG. 6-4 LANDSLIDE RISK EVALUATION PROCEDURE

For the case of Repentigny, the second method will be used as follows:

Probability factor = actual risk grade ÷ 12 = 11 ÷ 12 = 0.92

Assuming that the value of elements at risk is $5 million:

Risk value = 0.92 x 5 = $ 4,6 million.

Mitigation measurements can be adopted as shown in section 3.3 and the case history in 3.4. If protective measurements can reduce the risk grade to 6, for example, then:

Risk value = (6 ÷ 12) x 5 = $2.5 million

Reduction of risk value = 4.6 - 2.5 = $ 2.1 million

Therefore cost of protective measurements can be compared to the amount of $2.1 million.

Chapter: 7

CONCLUSION

Landslide, like any other hazard, is an interaction of nature and people. Hazard damages are increased by increase of degree of landslide and the increase of population and value of exposed structures. Therefore, by knowing the expected risk, damages of a natural hazard can be minimized through minimization of elements exposed to high risk.

Landslide can be triggered by natural factors or human-induced factors when they reach extreme values. Examples of natural factors include earthquakes, heavy rainfall and erosion from streams. Whereas examples of human-induced factors include excavation at toe of slope, mining and construction works.

On the other hand, landslide may be significantly controlled through mitigation measurements. These measurements include soil grading, drainage systems, and soil strengthening. Landslide case history showing a sucessful slope stabilization was introduced in section 3.4.

By studying the natural conditions and human activities in addition to site properties, landslide risk can be assessed. In this study, ranks from one to twelve were used to express the degree of landslide possibility. A simple model, to consider all human and natural factors, is introduced for the purpose of risk assessment. The risk assessment rank can be used for calculation of the expected damages. The value of damage will depend on the population and human activities at the risk area.

Publics and governments may use the risk assessment for decision on actions to be taken on present structures. It can also be used for future land use planning by avoiding the higher risk zones. Other agencies like insurance companies and civil defense authorities will be able to plan there business accordingly.

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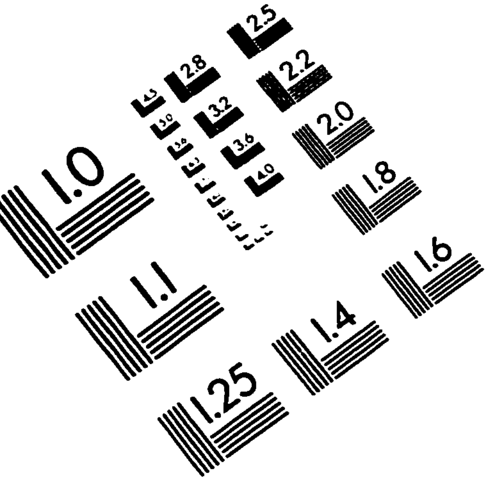
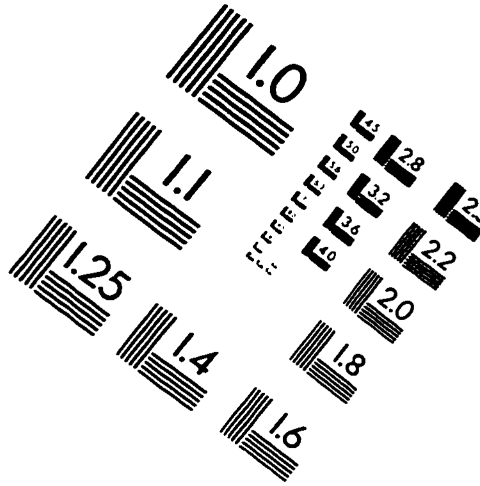
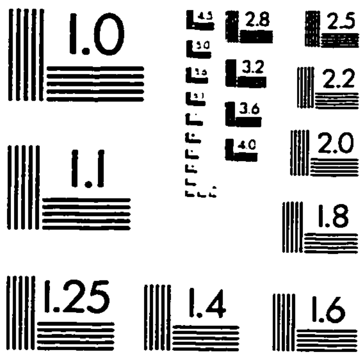
IMAGE EVALUATION TEST TARGET (QA-3)







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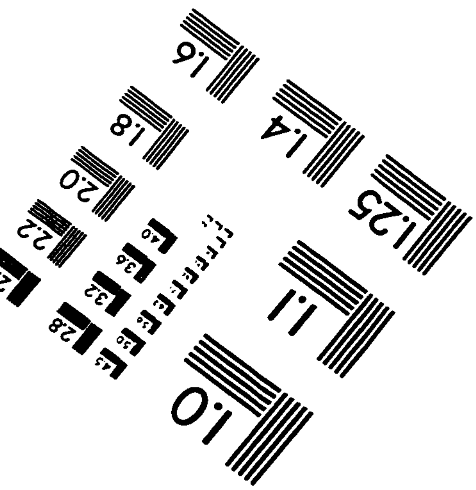

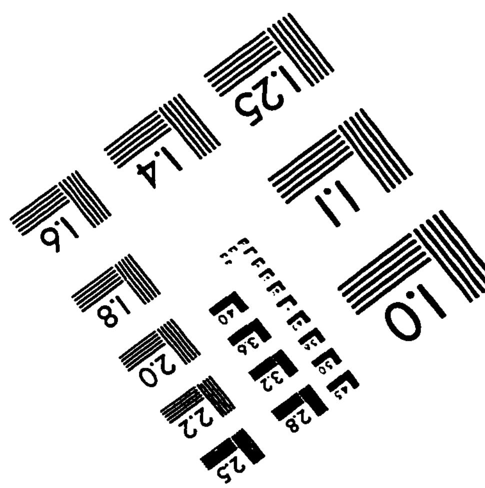

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Rochester, NY 14609 USA
Phone: 716/482-0300
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